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APRIL 1938.

ORDINARY MEETING.

22 February, 1938.

SYDNEY BRYAN DONKIN, President, in the Chair.

The PRESIDENT offered a cordial welcome to the President and members of the Institution of Electrical Engineers, who had been invited to be present that evening.

On the recommendation of the Council, the members present elected by acclamation as

Honorary Members.

His Majesty LEOPOLD III, K.G., King of the Belgians.
His Royal Highness GUSTAF ADOLF, DUKE OF SKÅNE, G.C.B., G.C.V.O., Crown Prince of Sweden.
Sir ROBERT ELLIOTT-COOPER, K.C.B., Past-President.
Sir FRANK EDWARD SMITH, K.C.B., C.B.E., D.Sc., LL.D., F.R.S.

The Council reported that they had recently transferred to the class of

Members.

JOHN BALLANTYNE CARSWELL, B.Sc. (*Glas.*).
BHUPALAM SURYANARAYANA CHETTI, M.A., B.Sc. (*Edin.*).
ROBERT MANFIELD FINCH.
WILLIAM EDWARD GELSON, M.Sc. (Eng.) (*Lond.*).
ROBERT ECKLIN MARRIOTT, B.Sc. (*Glas.*).
HARRY ABRAHAM SMITH, B.Sc. (*Cape Town*).
RICHARD WILLSHER WEEKES.

And had admitted as

Students.

GRAHAM ANDREWS-SPEED.
JAMES GRANT ARMSTRONG.
TONY ARRATOON.
ERIC DONALD AUSTIN.
DAVID REGINALD BATTERSON.
JULES STEPHEN BRABANTS, B.E. (*National*).
ALAN ARNOLD BROWN.
RICHARD BRISCO BULMAN.
KENNETH BUTTERS.
THOMAS CAIRNS.
THOMAS IAN CAMERON, B.Sc. (*Glas.*).
WILLIAM JAMES CAMERON, B.Sc. (*Glas.*).
DONALD ALASTAIR COODE.
BENJAMIN COOKSON.
HARRY CRONSHAW.
SHANTARAM MAHADEO DAHANUKAR, B.E. (*Bombay*).
FREDERICK JOSEPH DANIELS, B.Sc. (*Birmingham*).
IVOR AUSTIN GEORGE ELLISON.
BRIAN DOUGLAS FARMBROUGH.
GORDON PHILLIP WILLIAM FORREST.
ERNEST FINDLAY FROUD.
ALLAN SCOTT FURNESS.
DENIS SCOTT GARVIE, B.Sc. (*Aberdeen*).
LEONARD ARNOLD HAMEY.
ARTHUR GUY HARRISON.
WILLIAM YOUNGSON HUTCHISON.
GILBERT KEIGHLEY.
JAMES BERNARD KUIPERS.
GHULAM HYDER MOOSA KURESHI.
WILLIAM KERR LAUGHLIN, B.Sc. (*Belfast*).
GEORGE HAROLD LISTER.
McALISTER PENDER LONNON, B.A. (*Cantab.*).
DONALD LUMBARD, B.Sc. (*Bristol*).
ARTHUR JAMES STUART McLAREN.
IAN REID SMITH MACLEAN.
LAMBERTUS MEIJER, B.Sc. (*Witwatersrand*).
JAMES PEEBLES.
EDWARD FREDERICK PITTOCK, B.Sc. (Eng.) (*Lond.*).
ROBERT PRINGLE.
FREDERICK LOUIS ROBINSON.
WOLF SEFTON.
ARTHUR GEORGE SHELDON.
NOEL BRIAN SMETHURST.
IAN GRAEME STOREY.
KENNETH GEORGE STRATFORD.
SAM FLETCHER TOWNSEND.
GEOFFREY ARTHUR WHINCAP.
ERIC THOMAS WHITFORD.

The following Papers were submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5164.

"The Galloway Hydro-Electric Development, with Special Reference to the Constructional Works." †

By William Hudson, B.Sc. (Eng.), and
John Kenneth Hunter, B.Sc. (Eng.), MM. Inst. C.E.

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Introduction.

In preparing this Paper the Authors have been faced with the difficulty of doing justice, within the available space, to a subject which embraces such a great variety of constructional work. Articles dealing with various sections of the work carried out in Galloway have appeared from time to time in the technical press,[1] and, in order to avoid repetition, attention has here been devoted primarily to those special features, whether of design or construction, which are either novel in themselves or which involved the solution of problems not commonly encountered in general engineering practice. With this object in view those sections of the Paper describing the details of the individual structures have been curtailed to the minimum necessary for a proper understanding of the scheme,

† Correspondence on this Paper can be accepted until the 15th July, 1938. —Sec. Inst. C.E.

[1] *Engineering*, vol. 138 (1934, Part 2), pp. 267, 345, 429, 557, 639; vol. 142 (1936, Part 2), pp. 241, 297, 325, 383, 550, 604, 629, 687.

The Engineer, vol. 158 (1934, Part 2), pp. 366, 381, 410; vol. 162 (1934, Part 2), pp. 265, 283, 309, 337.

and in many instances it has been necessary to omit altogether specific mention of subsidiary works.

History.

The hydro-electric works belonging to the Galloway Water Power Company are situated in the south-west of Scotland and occupy the southern portion of the mountainous region which lies between the estuaries of the Clyde and the Solway.

In 1922 the Burgh of Ayr obtained parliamentary powers to undertake a partial development of the loch Doon catchment, but the project was never proceeded with, and the initiation of the larger and more comprehensive scheme which forms the subject of this Paper was due to two local residents, Major Wellwood Maxwell and Captain Scott Elliot, who, in 1923, discussed the matter in general terms with the late Colonel William McLellan, C.B.E., a partner in the firm of Messrs. Merz & McLellan. Later the firm of Sir Alexander Gibb & Partners was approached, and after making preliminary investigations they reported that the water-power resources of the district might be developed to the extent of providing the equivalent of about 20,000 kilowatts of continuous power.

The scheme was, at that time, faced with two difficulties. Firstly, as the south-west of Scotland is sparsely populated and purely agricultural in character, the potential demand for power in the surrounding districts was, and still is, very limited. Secondly, the physical characteristics of the area do not lend themselves to the economic construction of the large seasonal-storage reservoirs which would be necessary were it proposed to develop the scheme for the supply of an industry offering a relatively uniform demand throughout the year. The position became altered, however, on the passing of the Electricity Supply Act of 1926, and the subsequent construction of the National Grid made it practicable to tap hitherto undeveloped sources of power and to transmit the energy over long distances to established industrial centres. From a detailed consideration of the distribution of run-off and the physical features of the area, it was evident that the maximum economic use of the Galloway resources would be realized if the scheme were designed to meet peak-load requirements. This object could be the more easily attained since the winter rainfall is ordinarily much greater than that which occurs during the summer months, permitting the increased electrical demand during the winter to be met without having recourse to large seasonal-storage reservoirs. Accordingly, designs were prepared for the development in two stages of a total

of 102,000 kilowatts, to be operated on an annual load-factor of about 20 per cent.

The Power and Traction Finance Company secured an Act of Parliament in 1929 incorporating the Galloway Water Power Company with powers to construct five power-stations, together with the necessary reservoirs and other works. The Electricity Commissioners, who had signified their approval of the scheme when it was before Parliament, recommended during 1931 that the stations when constructed should be "Selected Stations" and that the entire output, after meeting certain small local requirements, should be taken by the Central Electricity Board for general distribution throughout the industrial areas of Central Scotland and North-West England. Plans for the South Scotland area were then adopted by the Central Electricity Board, who thereupon entered into an agreement with the Galloway Company for the purchase of the output from the projected power-stations.

The first contract was let in November, 1931, and the two power-stations comprising the first stage of the scheme were put into commercial service in March and May, 1935, respectively. The second stage was commenced in April, 1934, and the three additional power-stations which this embraced were completed in time to deal with the winter load of 1936-37.

The area which has been developed includes the entire catchment of the Galloway Dee (345 square miles); in addition, a further 50 square miles which lie to the north-west and form the catchment-area of loch Doon have been utilized by tunnelling through the low ridge which separates the two drainage-basins (Fig. 1, Plate 1).

Amenities Committee.

Provision was included in the Galloway Water Power Act for a Committee to be appointed by the Secretary of State for Scotland with powers to consider the plans of the works and to make such recommendations, having regard to the preservation of the local amenities, as they might think proper. The following gentlemen served on the Committee :—

Lord Hamilton of Dalzell, K.T., C.V.O., M.C.,
Sir John Milne-Home, D.L., J.P.,
Mr. H. O. Tarbolton, A.R.S.A., F.R.I.B.A.

Throughout the work the Committee maintained the closest co-operation with the Company and their Engineers, and were most helpful in the suggestions they put forward with a view to ensuring the suitability of the various works to their surroundings.

Geology.

The greater part of south-west Scotland is occupied by intensely-folded rocks belonging to the Ordovician and Silurian systems, resulting in a structure of great complexity. Within the Galloway district occur three extensive granite intrusions, those of Criffel bordering the Solway Firth, of Cairnsmore of Fleet in the centre of the district, and of loch Dee extending in a northerly direction to the southern end of loch Doon. A further small granite intrusion occurs to the east of loch Doon at Cairnsmore of Carsphairn. Great alteration has been produced around the margins of these granite masses, which are surrounded by a broad band of metamorphosed sedimentary rocks. In addition to the granite intrusions, a large number of dykes, consisting mainly of porphyrite, cut both the plutonic rocks and the surrounding sediments. One of these dykes cuts diagonally across the river Dee at Tongland dam, the metamorphic greywacke in the vicinity of the plane of contact presenting a soft and shaly consistency. The red porphyrite of which this dyke is composed was found to be much shattered and traversed by innumerable fine joints which made it impossible to excavate the rock in large blocks, although when crushed it produced an excellent concrete aggregate.

With the exception of the Clatteringshaws dam, which is located on the northern edge of the granite mass of Cairnsmore of Fleet, all the works in Galloway lie on the greywacke formation. In general this rock was found to be badly jointed and weathered near the surface, and deep foundations were necessary for many of the dams.

Hydrology.

Rainfall.

The climate of Galloway is mild and the rainfall comparatively heavy, varying from 40 inches on the coast to the south to over 90 inches on the high ground between loch Trool and loch Doon, where the hills rise to an altitude of over 2,700 feet. Records covering the district and extending over a considerable period were already in existence when the scheme was being prepared, and it was thus possible to estimate the annual run-off with considerable confidence. Since the Act was passed a number of new rain-gauges have been established within the catchment-area affected by the scheme, particularly at the higher elevations in the more remote parts of the area where previously observations were lacking. At the present time the Galloway Water Power Company has twenty rain-gauges under its own control, and in addition information is

available from nine independent observers who send in regular returns to the Meteorological Office. There is thus one gauge to every 14 square miles of catchment-area.

From the available rainfall-records the adjusted average values for the principal rain-gauge stations lying within or adjacent to the catchment were calculated for the standard 35-year period 1881 to 1915. From these standard values a rainfall-map (*Fig. 2*, p. 328) has been prepared by the Meteorological Office to show the distribution of rainfall over the area. From this map the mean rainfall for the standard period has been computed at 64·0 inches.

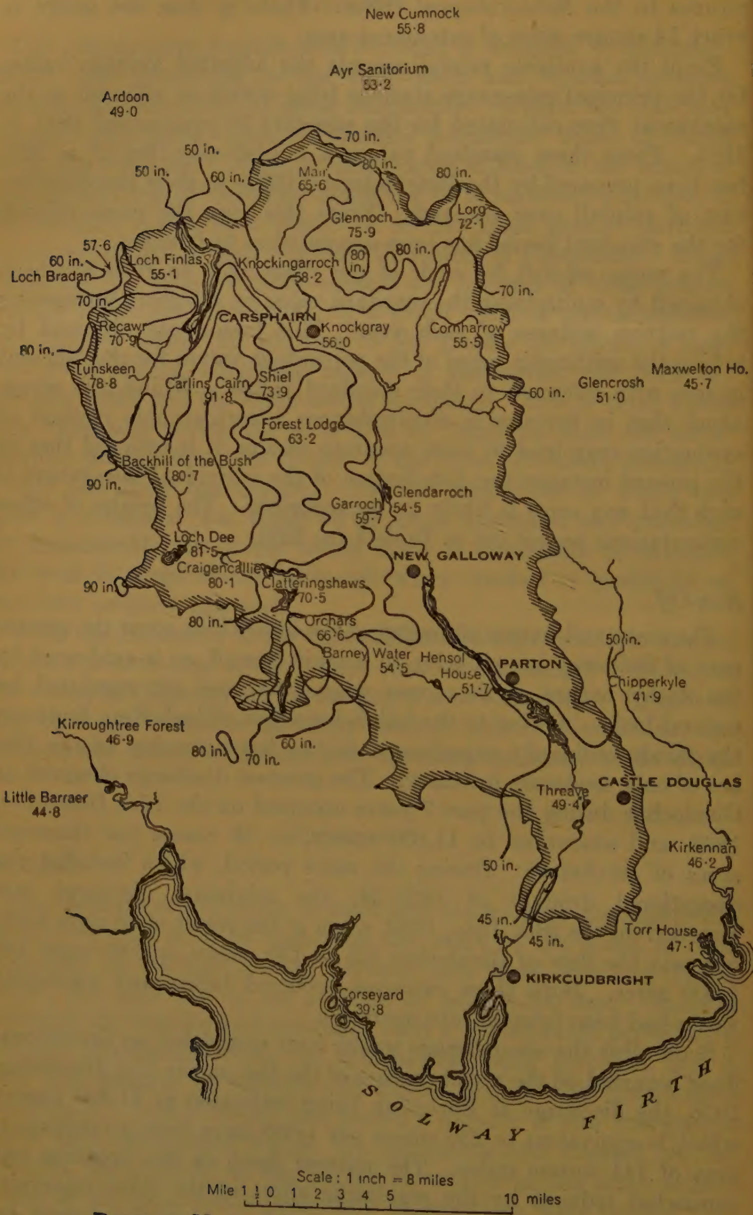
The mean rainfall for each of the years 1930 to 1936 has been obtained by multiplying the standard value by a factor representing the relative wetness of each year. This factor was obtained by taking the arithmetic mean of the percentage values for the different gauges with reference to their standard values. Whilst it is recognized that in favourable circumstances a cartographic method of evaluation may lead to more accurate results, it is believed that in the present instance the distribution of the individual rain-gauges is such that any error is likely to be negligible. The results of these computations are set out in Table I (p. 329).

Run-Off.

The steep and barren character of the hills which cover the greater part of Galloway is conducive to a rapid run-off, as is evidenced by the flashy nature of the rivers where they are not regulated by natural lochs. Owing to the balancing effect of loch Ken, however, the floods ordinarily experienced in the lower reaches of the Dee are of only moderate intensity. The greatest discharge observed at Glenlochar during the past 7 years occurred on the 17th December, 1932, and amounted to 11,800 cusecs, or 59 cusecs per thousand acres of catchment. During the same period, which included the exceptional drought of 1933–34, the minimum discharge was recorded on the 28th June, 1932, when as the result of 28 days without rain the flow of Glenlochar fell to 130 cusecs or 0·65 cusec per 1,000 acres. Both these extremes occurred before any regulating works had been brought into operation.

Soon after the second-stage works were completed an exceptional flood occurred on the upper reaches of the Dee on the 13th December, 1936, the discharge at Earlstoun being estimated at 21,800 cusecs, which is equivalent to 236 cusecs per 1,000 acres from a catchment-area of 144 square miles. The natural flood on this occasion was somewhat reduced by the regulating effect of the three reservoirs at Kendoon, Carsfad and Earlstoun, and to a smaller extent by the diversion into loch Doon of water from the Deugh to the extent

Fig. 2.



RAINFALL-MAP OF GALLOWAY FOR PERIOD 1881 TO 1915.

TABLE I.—MEAN ANNUAL RAINFALL OVER GALLOWAY CATCHMENT : INCHES.

	Standard period 1881–1915.	1930.	1931.	1932.	1933.	1934.	1935.	1936.
Combined area of Dee and Doon: 395 square miles . .	64·0 (100 per cent.)	73·2 (114·5 per cent.)	69·9 (109·1 per cent.)	71·8 (112·2 per cent.)	48·3 (75·1 per cent.)	69·6 (108·8 per cent.)	66·2 (103·4 per cent.)	66·5 (103·9 per cent.)
Doon area: 50 square miles	71·4 (100 per cent.)	78·9 (110·4 per cent.)	74·0 (103·6 per cent.)	79·3 (111·0 per cent.)	48·6 (68·1 per cent.)	81·4 (114·0 per cent.)	73·6 (103·1 per cent.)	71·6 (100·6 per cent.)
Dee area: 345 square miles	63·0 (100 per cent.)	72·5 (115·0 per cent.)	69·6 (110·6 per cent.)	70·9 (112·7 per cent.)	48·3 (76·7 per cent.)	67·8 (107·7 per cent.)	65·2 (103·4 per cent.)	65·8 (104·5 per cent.)

of about 500 cusecs. In spite of the consequent flattening-out of the peak, the recorded flood of 21,800 cusecs is in excess of that obtained from the curve for normal maximum floods recommended by the Institution Committee on Floods.[1] On the same occasion the maximum flood which passed Glenlochar, 15 miles downstream of Earlstoun, amounted, as a result of the regulating effect of loch Ken, to no more than 10,300 cusecs. The highest flood recorded at Glenlochar during the last 120 years occurred in 1872, when the estimated discharge of the river Dee was 28,000 cusecs.

Soon after the Act was obtained, a gauging station was established at Glenlochar bridge at the outlet from loch Ken. From the records obtained Table II has been computed showing the mean rainfall, run-off and losses over each of the 6 years 1930–1935.

TABLE II.—DEE CATCHMENT-AREA (ABOVE GLENLOCHAR BRIDGE); 311 SQUARE MILES.

	1930.	1931.	1932.	1933.	1934.	1935.	Average.
Mean rainfall : inches . .	74·3	71·5	72·7	49·6	69·5	66·7	67·4
Run-off : inches . . .	57·9	58·9	58·1	37·9	58·2	51·2	53·7
Total loss : inches . . .	16·4	12·6	14·6	11·7	11·3	15·5	13·7

The information at present available with regard to the intensity and distribution of rainfall on the 50 square miles of the loch Doon catchment is insufficient to allow of reliable estimates being made for the mean rainfall and losses.

Compensation-Water.

On account of the mills already established on the river Doon and the requirements of public health, the agreement incorporated in the Act provided that compensation-water should be released to the extent of 45 million gallons per day on six days a week, and to the extent of 40 million gallons a day on the seventh day. The provision is dependent upon the quantity of water abstracted by the Burgh of Ayr not exceeding a specified quantity, and the compensation-water is to be correspondingly reduced if this quantity should be exceeded. The requirements of the River Doon Fishery Board were met by the Promoters agreeing to release an additional 80 million gallons once in every 3 weeks, with the object of creating occasional freshets which would encourage the salmon to run. The total compensation-water required is equivalent to an average rate of

[1] "Interim Report of the Committee on Floods in Relation to Reservoir Practice," Inst. C.E., 1933.

48 million gallons per day, corresponding to a rainfall of 24·0 inches over the catchment, or 33 per cent. of the average annual rainfall.

In the case of the river Dee, the Promoters were more fortunate in that, with the exception of an artificial-silk factory at Tongland, no previous industrial use had been made of the water. The owners of this factory agreed to relinquish all claim to the river for power purposes, and accepted in its place a lump-sum payment together with the supply of electrical energy on favourable terms. The claims of the fishing interests were satisfied in the case of the Tongland works by the release down the fish-pass of 12 million gallons of compensation-water per day for the period March to October. It was further agreed that during this period additional water should be released to the extent of 30 million gallons per week. This additional supply is provided during two periods, one of 24 hours in the middle of the week and the other of 36 hours at the week-end. For the remaining months of the year the discharge down the pass is reduced to 5 million gallons per day. Similar arrangements were made in the case of the other dams constructed on the river Dee.

Method of Development.

The broad method of development will be clear from an inspection of Figs. 1 and 3, Plate 1. At the northern end of the district the principal seasonal-storage reservoir has been formed by raising the level of loch Doon 27 feet by the construction of a dam across its natural outlet. Water from this reservoir can be drawn off through a tunnel and discharged as required into the Dee down Carsphairn Lane, the name given to one of its upper reaches. During times of heavy run-off water can be diverted into loch Doon from a portion of the Dee catchment having an area of 30 square miles, which normally supplies one of the upper tributaries known as the Water of Deugh. The run-off from this area is intercepted by a system of catch-water conduits and weirs and passed into a free-flowing tunnel by means of which it is conveyed to an open aqueduct in Carsphairn Lane, which in turn is connected to the Doon pressure-tunnel. The steel pipe forming the connexion between this tunnel and the aqueduct from the Deugh is provided with a 5-foot diameter needle valve by which the natural flow of the Lane may be augmented by drawing water from both the Doon and the Deugh catchments. When this valve is closed the whole of the water intercepted by the Deugh aqueduct system is passed into the loch Doon reservoir, thus reversing the normal direction of flow in the Doon tunnel.

The first of a series of three power-stations is situated 9 miles below the draw-off valve in Carsphairn Lane. Known as the Kendoon power-station, it operates under an average net head of

150 feet, which has been obtained by the construction of dams across the Water of Deugh and the Water of Ken, the two principal tributaries which form the headwaters of the Dee. The Carsfad power-station is 1½ mile farther downstream and works under an average net head of 65 feet; immediately below it is the Earlstoun power-station which utilizes a further 67 feet. These three stations are normally operated together, and each is provided with its own head-pond to furnish the necessary daily regulation. The storage provided by these head-ponds is, however, small, and for seasonal regulation reliance is laid on the loch Doon reservoir.

A fourth power-station has been built at Glenlee about 2½ miles north-west of New Galloway. This station derives its water-supply from a separate catchment-area feeding the upper reaches of the Blackwater of Dee, a tributary stream which flows into loch Ken. A seasonal storage reservoir, known as Clatteringshaws loch, has been provided by damming the Blackwater of Dee and submerging an area of flat marsh land. The water thus impounded is diverted by means of a tunnel and pipe-line, having a combined length of nearly 4 miles, to the power-house on the banks of the Ken, where an average net head of 380 feet is utilized.

Below the Earlstoun power-station the river Dee enters a narrow lake, loch Ken, 10 miles in length. At the outlet from this lake, near Glenlochar bridge, a barrage of lifting sluice-gates has been provided by means of which the normal level of the loch can be raised for the purpose of providing weekly storage for the fifth and last power-station situated at Tongland on the estuary of the Dee.

Immediately above the village of Tongland the Dee flows through a narrow gorge. By damming this gorge it has been possible to utilize the greater part of the fall in the river between loch Ken and the tidal estuary, the average net head at the Tongland power-station being 106 feet.

For convenience the leading particulars of the reservoirs, dams, and power-stations are set out in Tables III, IV (pp. 334, 335), and V (p. 336) respectively.

TABLE III.—PRINCIPAL PARTICULARS OF RESERVOIRS OF GALLOWAY POWER WORKS.

Name of reservoir.	Storage capacity: millions of cubic feet.	Maximum draw-down: feet.	Spillway-level: O.D.	Water area at spillway-level: acres.
Loch Doon . . .	2,930	40	705	2,160
Kendoon	40	8	510	147
Carsfad	30	8	338	100
Earlstoun . . .	44	8	245	138
Clatteringshaws . .	1,250	40	585	1,020
Loch Ken . . .	320	4	148	2,090
Tongland	30	10	120	106

Description of Principal Works.

The more important works will be described in their natural sequence, starting at the northern end of the district at loch Doon.

Loch Doon Storage-Works.

Loch Doon provides the main seasonal storage for the upper three power-stations on the Dee, having an available capacity of 2,930 million cubic feet and a total draw-down of 40 feet. The original level of the loch has been raised by 27 feet, from 678·0 O.D. to 705·0 O.D., by the construction of the loch Doon dam across the natural outlet at the northern end. The total length of this dam (Figs. 4, Plate 1) is 980 feet, the main central portion consisting of a mass-concrete structure of the gravity type, slightly curved in plan. An arcade carries a 16-foot roadway over the dam. Flood-water is dealt with by means of a spillway having an effective length of 110 feet, in addition to which a battery of three siphons has been installed on the right flank. These siphons each have a discharging capacity of 850 cusecs, and come into operation successively, the inverts of the throats being at 706·5 O.D., 706·83 O.D., and 707·17 O.D. Each siphon terminates in a 5-foot 6-inch diameter Glenfield-Kennedy disperser. Compensation-water is released through a culvert provided in the base of the dam.

An interesting feature of the loch Doon dam is the fish-pass (Figs. 4, Plate 1), the upper portion of which consists of a series of chambers rising spirally inside a circular concrete tower standing within the reservoir. To allow for the varying level of the reservoir, alternate chambers are provided with a sluice-controlled orifice giving direct access to the loch. When the reservoir is drawn down below about 676·0 O.D., the main pass becomes inoperative, and, by opening the sluice-gate controlling the culvert through the dam, fish can enter or leave the loch by using the old outlet-channel.

In addition to the main dam, the raising of loch Doon involved the construction of a small dam across a low saddle on the eastern side of the loch in order to prevent water flowing over the watershed into Carsphairn Lane. Known as the Muck Burn dam, it has a length of 500 feet, and is of the earth-fill type with a concrete core-wall carried down to rock over the greater part of its length.

Doon-Deugh Tunnel Works.

The general purpose of the Doon–Deugh tunnels has already been mentioned, and their operation can be followed by reference to Fig. 3, Plate 1. Water from loch Doon can be drawn off into the tunnel leading to Carsphairn Lane through an intake constructed

TABLE IV.—PRINCIPAL PARTICULARS OF MAIN

Name of dam.	Principal function.	Type.	Maximum height of footway above river-bed : feet.	Total length along crest : feet.
Loch Doon .	Seasonal storage.	Gravity.	43	980
Muck Burn .	To close depression in watershed of loch Doon.	Rock-fill embankment with concrete core-wall.	15 (above original ground).	500
Deugh . .	To create head and provide daily storage.	Arch and gravity.	85	780
Ken . . .	To create head and provide daily storage.	Arch and gravity.	81	830
Blackwater .	To form intake for Kendoon aqueduct.	Arch and gravity.	50	610 (including intake).
Carsfad . .	To create head and provide daily storage.	Arch and gravity.	70	1,650 (including intake).
Earlstoun .	To create head and provide daily storage.	Arch and gravity.	67	700
Clattering-shaws.	Seasonal storage.	Gravity.	78	1,470
Tongland .	To create head and provide daily storage.	Arch and gravity.	72	977

mid-way along the eastern shore of the loch. The entrance to the tunnel is controlled by a roller gate with an opening 9 feet high by 6 feet wide. The mouth of the tunnel is protected by a removable screen sliding in vertical grooves, and this screen can be replaced by a bulkhead gate if it should become necessary to carry out repairs to the roller gate. The design of the intake-tower is such that when the roller gate is closed access to its downstream side may be obtained after the tunnel has been unwatered.

The Doon tunnel itself is straight in plan, has a total length of 6,372 feet, and falls towards Carsphairn Lane with a uniform gradient of 1 in 1107. It is designed for flow under pressure, i

DAMS OF GALLOWAY POWER WORKS.

Arch portion of dam.			Spill-way-level: O.D.	Normal maximum depth over crest: feet.	Particulars of spillways.	Normal maximum spillway-capacity: cusecs.
Length: feet.	Radius: feet.	Batter of down-stream face.				
—	—	—	705	3	Overfall spillway 110 ft. long; three siphons 5 ft. 6 in. outlet dia.	4,675
—	—	—	—	—	None.	—
356	220	3 to 1	509·67	3·33	Overfall spillway 326 ft. long.	7,300
220	165	4 to 1	510	3	Overfall spillway 393 ft. long.	7,300
144	125	4 to 1	—	—	None.	—
328	190	—	338	3	Overfall spillway 736 ft. long.	14,500
283	145	4·35 to 1	245	3	Overfall spillway 287 ft. long; also two floodgates each 17 ft. 6 in. by 23 ft. 6 in.	20,000
—	—	—	585	3	Overfall spillway 350 ft. long.	6,700
290	145	4 to 1	120	3	Overfall spillway 325 ft. long; also two floodgates each 25 ft. by 31 ft.	30,000

roughly horse-shoe-shaped in section, and has an equivalent diameter of 8 feet. The concrete lining of the walls and crown was specified to cover the rock points with a minimum thickness of 3 inches, the minimum average thickness permitted being 7 inches. The corresponding thicknesses in the case of the invert were 3 inches and 5 inches respectively.

The Deugh tunnel, which is of the free-flowing type, has a total length of 7,016 feet and a uniform gradient of 1 in 187; only the side walls and invert are lined with concrete. Its purpose is to enable a part of the run-off from an additional 30 square miles of catchment to be run into loch Doon for storage. The water from

Table V.—Principal Particulars of Power-Stations of Galloway Power Works.

Power-station.	Catchment-area: square miles.	Average net head: feet.	Consumption of water on full load at average head: cusecs.	Number of units and size: kilowatts.	Gross volume of turbine-room, including loading bay, above main floor level: cubic feet.	
					Total.	Per kilowatt.
Kendoon .	152	150	1,900	Two, 10,500	223,000	10·6
Carsfad .	171	65	2,600	Two, 6,000	196,000	16·3
Earlstoun .	194	67	2,500	Two, 6,000	196,000	16·3
Glenlee .	47·5	380	870	Two, 12,000 Two, 500	146,000	5·8
Tongland .	395	106	4,250	Three, 11,000 One, 250	420,000	12·6

Bow Burn is intercepted by a concrete weir and passed through an intake-chamber into a lined catch-water conduit which discharges into a forebay formed on the Water of Deugh. The supply to the Deugh tunnel is taken off from this forebay through a settling chamber furnished with a submerged orifice, which, in conjunction with the spillway provided by the weir, effects the necessary regulation of the supply to the tunnel.

At its downstream end the Deugh tunnel discharges into an open concrete-lined channel, the upper section being provided with a stepped invert. This channel is 300 feet long and terminates in an open concrete settling-tank provided with a scour-sluice and spillway. The downstream end of this tank forms a transition to a reinforced-concrete pressure aqueduct of circular section having an internal diameter of 14 feet 2 inches, sufficiently large to ensure that air would not be drawn downwards by the flow of water. This aqueduct also has a length of 300 feet, and is laid on a gradient of 1 in 5·3. At its lower end it is connected by a suitable transition to an 8-foot diameter riveted steel pipe, which in turn is connected to the portal end of the Doon tunnel. The steel pipe is built up of 8-foot strakes of ⅝-inch plate; it is stiffened by angle rings at 8-foot centres and is carried on concrete piers 48 feet apart.

At the point where the pipe crosses Carsphairn Lane a 6-foot-diameter branch is taken off, and on the end of this is installed a 5-foot-diameter needle valve fitted with a jet-disperser. The valve is manually operated from day to day in accordance with instructions received from the Control Engineer at Kendoon.

Kendoon Works.

Reference to Fig. 1, Plate 1, will show that the Water of Deugh, which has been previously joined by Carsphairn Lane, joins the

Water of Ken about $4\frac{1}{2}$ miles below the village of Carsphairn. A short distance upstream of this confluence the two streams pass through narrow wooded gorges, where they have been intercepted by the Deugh dam and the Ken dam. The water impounded by these dams floods the low saddle separating the valleys of the Deugh and the Ken and forms a single reservoir, a cut through the saddle being provided to facilitate, under conditions of maximum draw-down, the transfer of the flow from the loch Doon reservoir.

The Deugh dam (Figs. 5, Plate 1) consists essentially of a concrete arch dam closing the river gorge, the right bank of which forms one of the arch-abutments. The left abutment rests partly against the river-bank and partly against a tangential gravity dam which provides the necessary spillway-accommodation. The arch portion of the dam has a maximum height above river-bed level of 85 feet (measured to the footway) and a developed length of 356 feet. A 5-foot-diameter manually-controlled draw-off valve of the needle type, fitted with a disperser, is provided. To permit the needle valve to be overhauled an emergency bulkhead gate of a special roller design has been provided, by means of which the upstream end of the culvert can be sealed. The gate is normally kept in the raised position in a gatehouse on the top of the dam.

The tangential gravity section of the dam has an overall length of 371 feet, and provides a spillway having an effective length of 326 feet, the crest-level being fixed at 509·67 O.D., 4 inches below that of the adjacent dam on the Ken. The footway is carried on piers over the spillway. Training walls have been constructed so that the water discharged during floods is confined to a definite channel, the soft material overlying the rock has been removed, and all pockets of friable rock have been replaced or protected by concrete.

The Ken dam is similar to the Deugh dam. The upper portion of the right abutment of the arch is formed by a short length of gravity dam of non-overflow section, while the gravity dam on the left bank which forms the abutment provides the necessary spillway accommodation. As the surface rock on the left bank is of poor quality, the spillway-channel was lined throughout, and after dropping sharply to the river terminates in a concrete bucket designed to destroy the kinetic energy of the flood-water and prevent erosion of the river-bed. In order to distribute the flow down the channel in a uniform manner, guide-walls are provided at those points where sharp changes in direction occur. The design of this channel was based upon the results of model tests.

The water from the Kendoon reservoir is drawn off by a canal which takes off at the extreme end of the Ken dam on the left bank,

22

and roughly follows the contour of the hillside in a southerly direction. The canal has a bottom width varying from 7 feet to 10 feet and is lined throughout its length of 2,500 feet with 6-inch concrete slabs cast in place, the side slopes having a batter of $1\frac{1}{2}$ to 1. The canal is designed to deal with a variation in water-level of from 8 to 10 feet, and terminates in a forebay formed by damming the ravine through which runs the Blackwater Burn, a small tributary stream which joins the Water of Ken about $\frac{1}{2}$ mile below the Ken dam.

The earthen embankment on the downhill side of the canal has an outer slope of 2 to 1, and was formed from material excavated from the canal cut, the overlying loamy material having been previously removed from the ground-surface. When the canal was first put into service a certain amount of leakage occurred at several points along the toe of the embankment, and, although it was thought that this leakage might, if left alone, gradually disappear in the course of time, it was felt to be more prudent to staunch it at the outset. Accordingly at those points where the leakage was observed a waterproof lining was laid over the water-face of the embankment. This lining was formed of a 5-ply layer of felt and bitumen covered with a reinforced-concrete slab 4 inches thick, the toe of which was keyed into the rock through which the bottom of the canal had been excavated.

On refilling the canal it was found that the waterproof lining was effective in stopping the leaks which had first been observed, but as the water-level in the canal was gradually raised to the normal operating level further leaks began to appear at several other points. As it was most important that there should be no risk of any interruption once the power-station was put into service, a continuous steel sheet-pile core was driven through the bank into original ground over those portions of the length which were not already protected by the waterproof apron.

The dam on the Blackwater Burn consists of a central arched portion with a flanking gravity dam on either side, that on the left bank forming one side of the intake-chamber through which water is drawn into the pressure-aqueduct. The intake (*Fig. 6*) is provided with a sluice-gate of the free-roller type 13 feet 6 inches square. The gate, which is partially counter-balanced, is designed for operation against an unbalanced head of 31 feet, and is provided with rubber-covered steel stanching bars which bear against machined faces. Lifting of the gate is normally effected electrically by push-button control from the gatehouse, whilst lowering takes place under gravity under the control of a centrifugal brake. In addition to the manual control, closure of the gate is effected automatically when the velocity through the aqueduct exceeds the

normal maximum by 30 per cent., the tripping mechanism being operated from a paddle which is fixed to the crown of the aqueduct on the downstream side of the gate. Provision is also made for tripping the gate from the power-station by means of the supervisory control. Debris is prevented from passing into the aqueduct by a set of vertical screens having a gross area at full-reservoir level

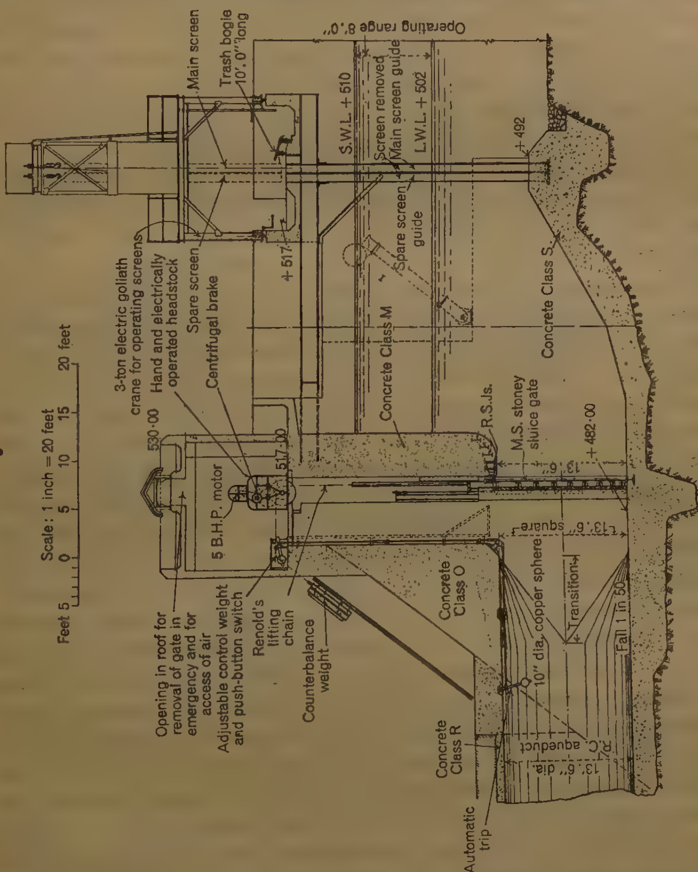


Fig. 6.

SECTION THROUGH INTAKE AT BLACKWATER BURN DAM.

of 900 square feet. These screens, which slide in double vertical grooves, are removable for cleaning and are handled by two electrically-driven hoists carried on a travelling gantry. The screen panels carry a close wire mesh made up of No. 8 and No. 12 gauge wire spaced respectively at 1¼-inch horizontal and ⅝-inch vertical centres for the purposes of excluding smolts—young salmon 5 to 6 inches in length on their first passage to the sea. At those times

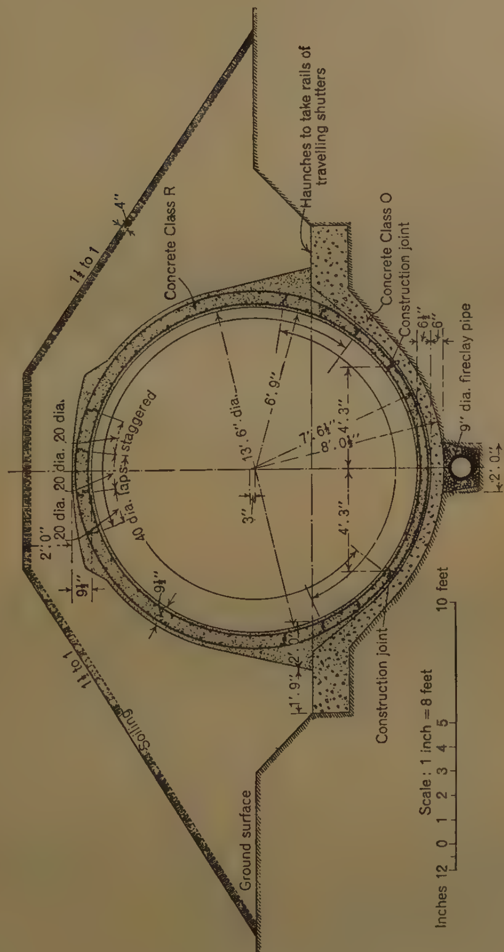
of the year when it is not necessary to protect the smolts, the fine screens are replaced by others of a coarser mesh.

Separate provision has been made at the intake for the passage of smolts in a downstream direction. As the water-level at the intake is subject to considerable variation, the upper portion of the pass was constructed in the form of a closed shoot, hinged about its lower end, the upper end being supported by a float which allows the inlet to the swivelling arm to be maintained at a constant level below the water-surface. The lower portion of the smolt-pass consists of a series of concrete chambers giving access to the natural bed of the Blackwater Burn. The normal discharge of the pass under full-reservoir conditions is approximately 17 cusecs.

The upper part of the pressure-pipe which connects the Blackwater intake with the power-station is a reinforced-concrete aqueduct (*Fig.* 7) having a diameter of 13 feet 6 inches and a length of 2,830 feet. This aqueduct terminates in a steel surge-tower 50 feet in diameter and 85 feet high. The maximum static head at the lower end of the aqueduct is 110 feet, and the maximum working stress in the circumferential steel was kept down to 11,000 lb. per square inch. The pipe leading from the surge-tower to the power-station is of riveted steel plate, its upper portion being 13 feet 6 inches in diameter. It is carried on concrete piers at 48 feet centres, movement due to temperature-changes being taken up by sleeve-type expansion-joints designed to deal with a total range of 100° F. A short distance upstream of the power-station the pipe bifurcates into two 10-foot-diameter branches which supply the turbines. These two branches are provided with hydraulically-operated butterfly valves, which can be opened or closed either by direct manual operation or by push-button control from the power-house. An overspeed-paddle trip is also provided to close the valves in case of accident. Immediately downstream of these valves the branch-pipes are reduced to 9 feet 6 inches in diameter and are buried and encased in concrete. The longitudinal seams were shop-welded electrically before delivery, whilst the circumferential site joints were riveted with external butt-straps. After complete erection the steel pipes were hydraulically tested, the upper section to a maximum of 140 lb. per square inch, and the branch pipes to a maximum of 165 lb. per square inch.

The Kendoon power-station contains two 10,500-kilowatt units of the vertical-shaft, single-floor type. The substructure of the power-station is of mass concrete suitably reinforced, and in it are embedded the plate-steel spiral casings of the turbines and the draught-tube liners. Special precautions were taken during the pouring of the concrete around the spiral casings to ensure that no voids were

Fig. 7.



TYPICAL CROSS SECTION OF KENDOON AQUEDUCT.

formed, and all points at which air might be trapped were afterwards completely grouted.

Carsfad Works.

The natural gradient of the 2-mile stretch of river which lies below the Kendoon power-station has been turned to account by the con-

struction of a dam which ponds back the water to 338·0 O.D., 1 foot below the level of the concrete sill controlling the tailrace-level at Kendoon. The dam (Figs. 8, Plate 1, and *Fig. 9*) is of combined arch and gravity type. It has a maximum height from the river-bed to the footway of 70 feet, and produces an effective head on the power-station of 65 feet. A view of the dam during construction is shown in *Fig. 10*. The draw-down is normally restricted to 2 feet, and the power-station operates in conjunction with that at Kendoon, where greater storage is available. In an emergency, however, the Carsfad head-pond can be drawn down to a maximum extent of 8 feet.

The discharge of flood-water at Carsfad has been entirely dealt with by spillways, and to obtain the necessary length the arched portion of the dam has been utilized as well as the gravity dam on the right bank. The combined effective length of the spillways thus provided is 736 feet, and with the designed head of 3 feet over the crest they are capable of discharging about 14,500 cusecs. Owing to the poor quality of the surface rock, the spillway-channels were mostly lined with concrete and steps were taken to ensure that the banks and river-bed would not be subject to water flowing at high velocities. The design of these channels and protective works was again based upon the results of extensive model-experiments.

To meet the requirements of the Scottish Fishery Board a fish-pass has been provided which will allow the fish to travel either upstream or downstream. The pass takes off from the river-bed close to the point where the turbines discharge into the tailrace, and comprises thirty-five steps divided by resting-pools into four flights. The entrance to the power-station intake is protected by a series of vertical screens of similar construction to those employed at Kendoon.

The power-station accommodates two vertical units of 6,000 kilowatts each, supplied from the forebay by two reinforced-concrete pipes, each of 13 feet 6 inches diameter and of similar construction to the aqueduct at Kendoon. The intake is controlled by sluice-gates of the free-roller type, each 10 feet wide by 14 feet high, which are normally operated from the control-room at Glenlee by means of the remote supervisory control but can also be operated locally by electric push-button control or by hand. As at Kendoon, the gates are partially balanced, and on being tripped either by hand or by means of the automatic overspeed-paddle, close by gravity. Filling of the turbines is effected by cracking open the main intake-gates, the initial opening being restricted to 2 inches by means of a limit-switch.

Since the Carsfad power-station is of the remote-controlled type

Fig. 9.

GENERAL VIEW OF CARSFAD WORKS.

Fig. 10.

CARSFAD DAM UNDER CONSTRUCTION.

5164.

Fig. 14.

TONGLAND DAM, FISH-PASS, FLOODGATES, AND SPILLWAY.

Fig. 19.

TONGLAND POWER-STATION.

no accommodation for any permanent staff is provided, and it was possible, therefore, to make it extremely simple and compact.

Earlstoun Works.

Between the tail-race of the Carsfad power-station and the head of loch Ken there is a difference of level of about 120 feet, rather more than half of which occurs in the 2-mile reach of river which lies below Carsfad. Advantage was taken of the existence of a rocky gorge in the bed of the Dee at Earlstoun to construct an arch dam very similar to that at Carsfad. The dam (Figs. 11, Plate 2), in conjunction with a headrace-canal which extends along the right bank for a distance of 1,200 feet, provides a gross head at the power-house of 69 feet.

At Earlstoun flood-water is dealt with by spilling over the crest of the gravity dam which forms the right abutment of the arch, in addition to which two large floodgates, each having a clear opening of 17 feet 6 inches and a vertical travel of 25 feet, are provided. The floodgates are of the free-roller type and are fully counterbalanced. Automatic float control is provided for opening the gates in the event of the reservoir rising above a pre-determined level, and is set so that both gates open 2 feet when there is a depth of water of 18 inches over the crest of the spillway, the opening increasing to 4 feet 6 inches should the reservoir-level rise a further 3 inches. For greater openings the gates are operated under manual control. As at Carsfad, the quality of the rock was such that it was considered necessary to line all spillway-channels and to provide them with energy-dispersing buckets.

The intake-gates, fish-ladder, turbine-pipes, and power-house are similar to those constructed at Carsfad. Like that at Carsfad, the power-station is remote-controlled from Glenlee.

Tongland Works.

Although in following the course of the river Dee in a southerly direction the Glenlee power-station would be passed soon after leaving Earlstoun, it is more convenient in describing the scheme to deal next with the Tongland power-station.

In order to increase the very limited regulation provided by the Tongland reservoir, the level of loch Ken has been raised by a barrage of lifting gates at Glenlochar. The dry-weather level of the loch is about 142·0 O.D., although in times of heavy flood it rises to 150·0 O.D., or even higher. The barrage is designed to raise the normal level of the loch to 148·0 O.D., and in the top 4 feet provides 320 million cubic feet of additional storage.

The six gates of the barrage (Fig. 12, Plate 2) are of the fully-balanced free-roller type of 45 feet clear span, three of them being 10 feet deep and the other three 9 feet deep. The piers in which the gates slide are of reinforced concrete, and are surmounted by a lattice-girder bridge which carries the lifting gear. The gates can be operated either by hand, by electric push-button control from the bridge, or by remote supervisory control from Tongland. In addition, float-operated control-equipment is provided which causes the gates to lift in succession in the event of the loch rising above a pre-determined level; a limit switch restricts the opening of the first two gates to lift, in order to prevent a too rapid increase in the flow of the river. A fish-pass has been incorporated in the centre of the barrage to allow the passage of fish when the gates are closed.

The Tongland dam (Figs. 13, Plate 2, and *Fig. 14*, facing p. 343) is generally similar to those already described at Kendoon, Carsfad, and Earlstoun. It consists in the main of a horizontal arch which closes off the main river-gorge and a tangential gravity dam on the left bank. The latter is returned into the hillside across a depression which once formed the old river-bed before the Dee took to its present and deeper channel, and is provided with two large flood-gates (*Fig. 15*), each having a span of 25 feet, similar to those at Earlstoun.

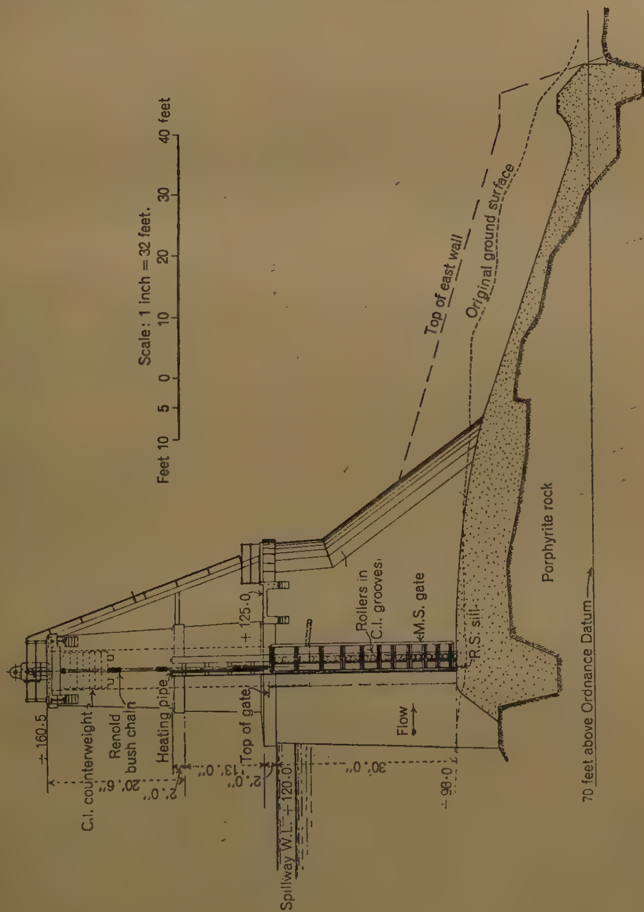
The deserted river-bed (*Fig. 14*, facing p. 343) has been utilized as a by-pass channel down which the flood-water discharged from the dam is returned to the river. Ordinary floods are dealt with by means of a spillway-channel excavated along the flank of the hill forming the eastern boundary of the reservoir. The downstream end of this concrete-lined channel, which is capable of discharging 6,000 cusecs with a head over the crest of 3 feet, is extended in the form of an apron and terminates in a bucket of similar design to those adopted in the case of the upper dams. The natural depression which lies below the flood-gates and spillway-channel has been utilized as a cushion-pool by the construction of a small concrete weir across its downstream end.

At this point it is convenient to digress in order to describe certain events which followed the closing of the temporary openings in the dam and the diversion of the river down its ancient channel.

Below the cushion-pool weir to which reference has been made, the old river-bed extends for a distance of about 60 yards and curves round to meet the present channel. The sedimentary rocks in the vicinity have been subjected to intense folding and have undergone great changes as the result of contact with the porphyritic dyke which here runs diagonally across the valley. Although it was known that the surface rock in the locality was for the most part

of very poor quality, it was anticipated, having regard to the relatively small drop of 20 feet from the toe of the cushion-pool weir to the existing river bed, that no serious erosion would take place. That confidence was, however, unjustified. After the closing of the last temporary opening through the dam the river was diverted down the floodgate-channel on the evening of the 20th October,

Fig. 15.



TONGLAND DAM : SECTION THROUGH FLOOD-GATES.

1934, a discharge of 1,700 cusecs being passed under the control of Glenlochar barrage. The following morning it was observed that as the result of this very moderate flow serious erosion of the river-bank had commenced at the point where the two channels met. It was at once apparent that retrogression was proceeding at a rapid

rate and would, if allowed to go on unchecked, eventually result in the destruction of the concrete weir at the south end of the cushion-pool, which pool in turn was used to protect the aprons of the spillway and floodgates. The situation called for drastic measures, and it was decided to commence immediately the removal of the concrete plug in the base of the arch dam with the object of turning the river back into its normal channel. Fortunately only a 3-foot thickness of concrete had been placed in the temporary opening when this decision was taken, but it took 7 days' continuous work to remove the plug, which contained about 30 cubic yards of concrete. Meanwhile, the situation was aggravated by floods which reached a peak in excess of 10,000 cusecs and rapidly eroded away the soft rock of the channel until the whole foundation of the cushion-pool weir was exposed. It was fortunate that this weir had been provided with an apron with a slightly upturned toe, which effectively prevented any undercutting and stopped further erosion.

After the removal of the concrete plug and the restoration of the river to its old bed, remedial measures were undertaken. These were based essentially on reconstructing the weir in such a way as to ensure the safe dissipation of the greater part of the energy of the flood-water. The design adopted was founded upon the results of model tests and is shown in Fig. 16, Plate 2.

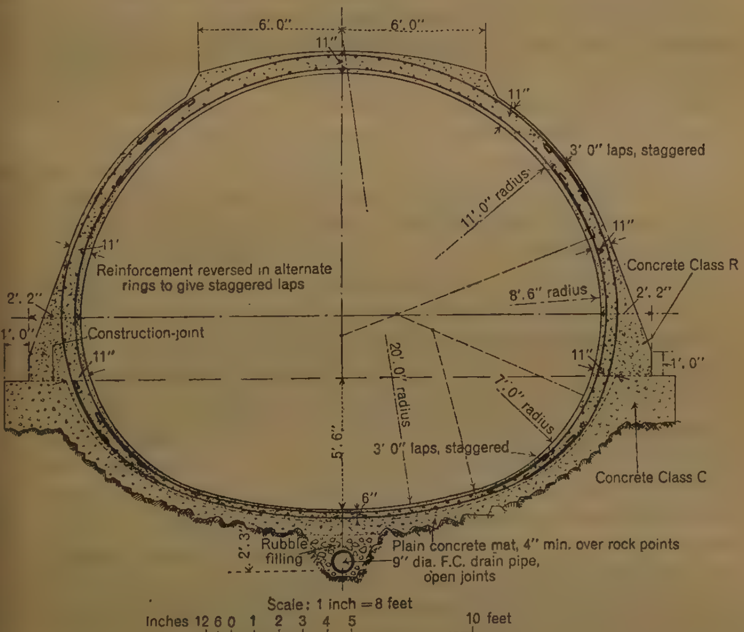
For power purposes the water is drawn off from the Tongland reservoir through an intake-chamber excavated on the right bank immediately upstream of the arch abutment. The intake is provided with fixed screens supported on a reinforced-concrete and steel structure and gives access to a tunnel 480 feet long which forms the upper portion of the aqueduct. The flow is controlled by a sluice-gate measuring 20 feet by 18 feet.

The main portion of the aqueduct is of reinforced concrete, and has a total length from the tunnel-portal to the surge-tower of 3,335 feet, the upper section having an equivalent diameter of 20 feet, while over the lower 975 feet the equivalent diameter is reduced to 18 feet 6 inches. One of the most interesting features of the aqueduct is its shape, which, as will be seen from *Fig. 17*, is a flattened circle, proportioned so as to produce under the varying internal pressure an approximately uniform circumferential stress not exceeding 12,000 lb. per square inch in the steel reinforcement. Longitudinal steel is provided to the extent of one-third of the circumferential steel.

The aqueduct terminates in a surge-tower of unusual proportions and design (Figs. 18, Plate 2, and *Fig. 19*, facing p. 343). It consists of a steel tank having a diameter of 100 feet and a depth of 49 feet 6 inches, the former dimension being fixed by considerations of

incipient stability. The tank is carried on a massive reinforced-concrete substructure, and is provided with an internal overflow-pipe terminating in a large bellmouth, the lip of which is at the same level as the spillway of the dam (120·0 O.D.). Beneath the surge-tower the aqueduct trifurcates, the three branch-pipes being controlled by butterfly valves having a diameter of 11 feet 6 inches, which are accommodated in a separate valve-house adjacent to the surge-tower and are operated by oil servo-motors.

Fig. 17.



Cross Section of Tongland Aqueduct.

The power-house building itself is of similar construction to those employed for the upper power-stations, but, in addition to the accommodation provided for switchgear and auxiliaries, office and store accommodation is furnished on a more generous scale.

The power-house being located on tidal waters, the head on the turbines is reduced by about 6 feet at high spring tides. Investigation showed, however, that increasing the maximum head by 5 per cent. by deepening the river-bed could be justified on economic grounds, and this object was attained by the removal from the tail-race of about 15,000 cubic yards of rock.

Glenlee Works.

The Glenlee power-station derives its water-supply direct from a seasonal-storage reservoir at Clatteringshaws on the Blackwater of Dee, with which it is connected by a tunnel and high-pressure pipe-line.

The Clatteringshaws dam is of the plain gravity type and is slightly curved in plan (Figs. 20, Plate 2). It has a total length of 1,470 feet and a maximum height above river-bed level of 78 feet. The footway is carried upon a series of arches, and flood-water is discharged over the crest by a central spillway having an effective length of 350 feet. The foundation rock consists of sound granite, practically free from important fissures, and little excavation was necessary.

The tunnel-intake (Fig. 21, Plate 2) was constructed in the form of a circular tower of heavily-reinforced concrete, the entrance to the tunnel being controlled by a sluice-gate of robust construction, having a clear span of 8 feet 6 inches and a height of 15 feet. The gate is designed for both opening and closing against the full head of 68 feet over the sill, and no separate filling sluice is provided. Control is effected in a similar manner to that already described in the case of the upper stations on the Ken. An approach-channel excavated in the bed of the reservoir and terminating in a forebay in front of the intake-tower allows a maximum draw-down of 40 feet.

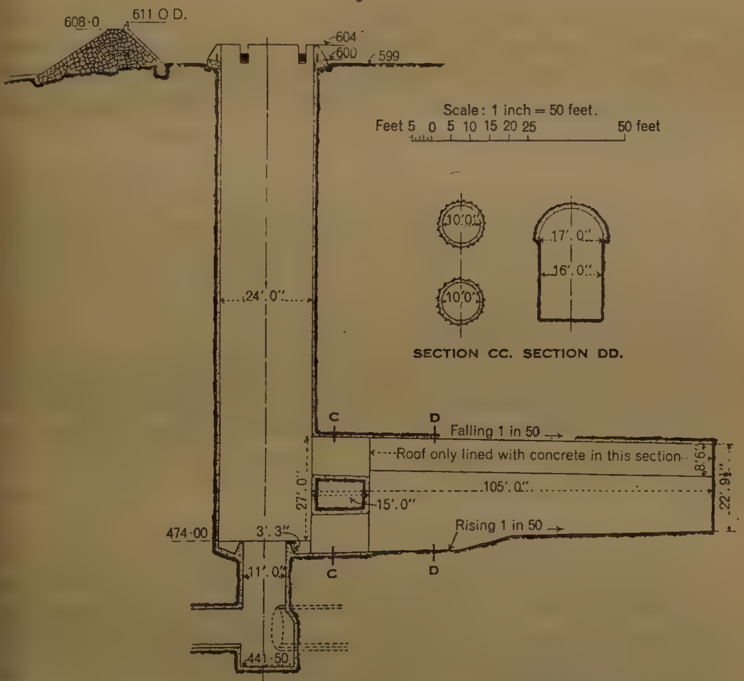
A noteworthy feature of the draw-off arrangements for the tunnel is the provision in the approach-channel of a series of concrete piers, between which steel stop-gates can be inserted. These gates permit the screens and forebay to be dewatered without the necessity of drawing down the reservoir to the invert-level of the channel, and provide a reserve of stored water for operating the plant immediately on the completion of any inspection or repairs to the tunnel-intake.

The tunnel has a total length of 18,988 feet and an equivalent diameter of 11 feet. It is concrete-lined throughout and over the greater part of its length is constructed on a gradient of 1 in 350, the gradient of the downstream section being increased to 1 in 100. At about its mid-point the tunnel passes under the Craigshinnie Burn, and the opportunity was taken of intercepting this stream and leading the water into the tunnel through the shaft which was used during the driving operations. The specification required that in the walls and crown no point of rock should come within 3 inches of the finished face of the concrete, and that the minimum average thickness of the lining should not be less than 8 inches. The corresponding figures for the invert were 2 inches and 5 inches respectively.

To provide the necessary hydraulic regulation for the turbines, a

concrete-lined surge-shaft (*Fig. 22*) was constructed as close to the portal end of the tunnel as the configuration of the ground would permit. The upper end of this shaft opens out into a small pond into which water is spilled during heavy upward surges. The pond is proportioned to deal with the tripping of the full station-load under conditions of maximum flood-level in the reservoir. Contrary to usual practice, the shaft has been designed to deal with the surges produced by suddenly throwing full load (24,000 kilowatts) on to

Fig. 22.



SECTIONAL ELEVATION OF GLENLEE SURGE-SHAFT.

the station under conditions of maximum reservoir draw-down, and to enable the resulting demand for water to be met as economically as possible a large horizontal gallery was driven into the hill near the bottom of the shaft.

The last 160 feet of the tunnel adjacent to the portal is lined with welded steel pipes having a diameter of 9 feet 6 inches and a plate-thickness of $\frac{7}{16}$ inch designed to take the full bursting pressure. Flow through the tunnel is controlled by the installation at the portal of a 10-foot butterfly valve operated by a hydraulic servo-

motor cylinder deriving its supply from the tunnel itself. Normally the valve is operated under balanced conditions, to provide which a by-pass valve is fitted, but under emergency conditions the valve can be closed against the full pressure, either through the medium of an overspeed-paddle trip or by push-button control from the power-house.

The pipe-line has a total length between the portal and turbine-valves of 1,427 feet, and ranges in diameter from 8 feet 9 inches at the upper end to 8 feet 4 inches at the main anchorage above the power-station, where it bifurcates into 6-foot diameter pipes which supply the two main turbines. It is of electrically-welded steel construction and has been designed on a very conservative basis. From considerations of stiffness the minimum plate-thickness was fixed at $\frac{1}{2}$ inch, the thickness where this figure is exceeded being determined by the use of a maximum working stress in the full plate of 13,500 lb. per square inch after deduction of a corrosion-allowance of $\frac{1}{8}$ inch. The efficiency of the longitudinal welded seams was ascertained by tests to be not less than 100 per cent., the material of the pipe being mild steel with an ultimate tensile strength of 64,000 lb. per square inch. The maximum static head at the lower end of the pipe-line is 410 feet, whilst the maximum working head resulting from the rejection of the whole of the power-station load is 500 feet. All pipes were tested at the manufacturer's works to a pressure 50 per cent. in excess of the maximum working pressure appropriate to their position. After erection the pipe-line was tested as a whole to a pressure which was equivalent to $22\frac{1}{2}$ per cent. in excess of the maximum working pressure at the bottom, and approximately 75 per cent. in excess of that at the portal end.

The welded longitudinal joints are provided with an external cover-strap, while riveting was adopted for those circumferential joints which had to be made on site, the butt-joint between the pipes being welded on the inside and the cover-straps caulked externally. The pipe is supported at 48-foot centres by sliding bearings on concrete piers, and sleeve-type expansion-joints are provided immediately below each anchorage. Two 6-foot-6-inch diameter guard-valves of the butterfly type are installed on the downstream side of the bifurcation, with the object of permitting one of the power-house units to be run in the event of it becoming necessary to overhaul the turbine-valve of the other. These guard-valves are arranged for manual operation under balanced-pressure conditions.

The power-house (Figs. 23, Plate 2) contains two main vertical-shaft units having an output of 12,000 kilowatts each, together with two auxiliary horizontal-shaft units of 500 kilowatts capacity. The

admission of water to the main turbines is controlled by cylindrical balanced valves.

As the Glenlee power-station was sited at some distance from the river Ken it was necessary to excavate a tailrace-channel 900 yards in length. At its downstream end the tailrace is provided with a low weir which ensures that the bed of the channel is kept flooded when the station is shut down, thus avoiding the unsightly appearance which it would otherwise present at such times. The weir is surmounted by a low screen to prevent the entry of salmon into the tailrace.

Fish-Passes.

The local proprietors in Galloway attached considerable importance to the fishing, and measures had to be taken to secure the preservation of the Dee and the Doon as fishing rivers. The arrangements which have been adopted to secure the passage of fish past the dams and power-stations have all been carried out to the approval of Mr. W. J. M. Menzies, the Inspector of Salmon Fisheries of Scotland, and have proved very successful. The Galloway Water Power Company were fortunate in obtaining the services of Mr. W. L. Calderwood, I.S.O., F.R.S.E., formerly Inspector of Salmon Fisheries, whose special knowledge of the habits of salmon and experience of fish-passes in many parts of the world proved of the greatest assistance.

The Galloway Water Power Act provided that fish-passes should be constructed at the dams at Tongland, Earlstoun, and Carsfad, in addition to which a pass for the descent of salmon fry was to be provided at Kendoon, together with fish-hatcheries on both the Dee and the Doon. On the latter river a fish-pass has been substituted for the hatchery originally contemplated, and the question of a hatchery on the Dee is at present in abeyance. The pass on the river Doon, which is of novel design, has already been described (p. 333), whilst further details concerning the design of fish-passes will be found in the manuscript of the Paper filed in the Institution Library. Brief mention will, however, be made here of the experiments carried out at Tongland in connexion with the passage of smolts through the turbines.

The question of possible injury to the smolts falls under the two heads of mechanical injury as the result of contact with the moving buckets of the runner, and internal injury consequent upon the sudden change of pressure in passing through the turbine. It was believed that in the case of machines having the physical dimensions of those installed at Tongland, where the water-passages are

relatively large, the risk of mechanical injury would be slight; it was therefore in the direction of determining the effect of rapidly-changing pressures that experimental information was sought. When the Tongland power-station is on full load a fish in passing from the open forebay to the tailrace is subjected to a pressure which in a period of 3¾ minutes gradually increases from about 10 lb. per square inch at the tunnel-intake to 30 lb. per square inch at the base of the surge-tower. During the following 11 seconds the pressure further increases to 45 lb. per square inch, which is attained at the entry to the spiral casing, whilst in the next second the passage through the turbines is effected and the pressure drops to a partial vacuum of 9 lb. per square inch below atmospheric pressure, to which it is gradually restored in the succeeding 8 seconds in passing through the draught-tube to the tailrace. In the face of these rather unpromising conditions, a series of experiments was carried out in which a number of fish were imprisoned in a tank and subjected to a complete cycle of pressure-variations in simulation of those which would obtain in passing from the forebay to the tailrace at Tongland. The experiments were very successful, in that they showed that no harmful effects to the fish resulted from their being subjected to pressure-changes of the magnitude employed, and in the light of this experience it was agreed that the smolts should be allowed to pass through the Tongland power-plant subject to the results proving satisfactory. Experience in this respect has so far been very favourable, and it is only infrequently that any stunned or injured fish are seen. Only a proportion of the smolts find their way to the sea by passing through the turbines, the majority using the fish-pass, in which they may be seen in large numbers during the migratory period.

The pool type of pass has been exclusively adopted in Galloway, and, with the exception of the fish-pass at Tongland power-station and the smolt-pass at Kendoon, where the more usual over-fall weir is employed, the connexion between adjacent chambers takes the form of a submerged orifice. Although passes of the former type, in which the water is spilled over the dividing walls between the chambers, are more easily regulated and are generally cheaper to construct, it was decided to adopt the submerged orifice for the sake of the improved conditions which would be obtained. Where the fish-passes give access to a reservoir in which the water-level is subject to fluctuations, a series of float-controlled sluices is provided, by means of which the higher chambers of the pass are shut off in succession as the reservoir-level falls.

At Kendoon, owing to the smaller physical dimensions of the turbines in the power-station and the higher head under which they

operate, it was decided to exclude the smolts from entry to the power-plant by fitting special fine-mesh screens at the forebay. Accordingly a smolt-pass has been provided with the object of allowing the young fish from a possible future hatchery on the upper reaches of the Dee to travel downstream on their first migration to the sea.

Apart from certain special cases where mechanical elevators have been adopted, as at the Baker River dam in the United States, it is believed that the fish-passes at Tongland, Earlstoun and Carsfad, which have heights of 70 feet, 70·5 feet and 74 feet respectively, are amongst the highest which have been constructed. In designing these passes the vertical lift between successive resting-pools has been limited to about 20 feet, and, provided that this distance is not greatly exceeded and that the pools are adequately dimensioned, there does not appear to be any reason why fish-passes of considerably greater total height should not be successfully constructed.

The pass at Tongland dam was the first to be put into service, and its operation was watched with a good deal of interest. Spring fish were already in the river at the time (April, 1935) and it was soon evident that they were experiencing no difficulty in finding the entrance to the pass and making the 70-foot ascent to the reservoir. On a number of occasions when the pass has been shut down for inspection between fifty and a hundred fish have been counted in the various pools and chambers.

Use of Models in Designing Hydraulic Works.

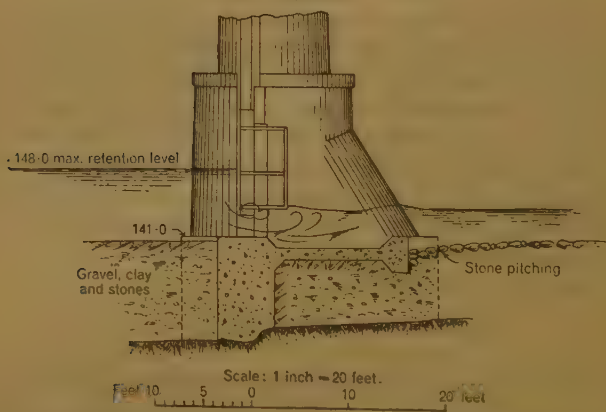
Throughout the design and construction of the Galloway works extensive use was made of models to determine the most satisfactory form of those hydraulic structures for which it was felt that ordinary engineering experience offered an insufficient guide. During the earlier stages the experimental work was carried out mainly in the Hydraulic Laboratory of University College, London, but later it was found preferable, owing to the limitations imposed on the quantity of water which could be handled in the college laboratory, to construct a testing flume on the site where larger-scale models could be used.

One of the first problems to be dealt with was the evolution of a suitable sill and protective apron for the barrage at Glenlochar, where the underlying rock in the river-bed is covered with a layer of coarse gravel and boulders. After a large number of designs had been tried with a model on a scale of $\frac{1}{4}$ inch to 1 foot, the arrangement shown in *Fig. 24* (p. 354) was finally adopted. A survey made

after the barrage had been in service for some months showed that no appreciable movement of the river-bed gravel had taken place.

The design of fish-pass chambers was also studied by means of models, the tests being principally directed to determining the best size and arrangement of orifices and baffles with the object of reducing turbulence and broken water to a minimum. Methods of dealing safely with the discharge of flood-water in spillway-channels were also dealt with experimentally, particular attention being given to the design of a suitable terminal bucket for the protective aprons which should safely dissipate the large amounts of energy represented by the release of flood-waters. The type of bucket adopted for the purpose is illustrated in *Fig. 25*, which shows the sill, protective

Fig. 24.



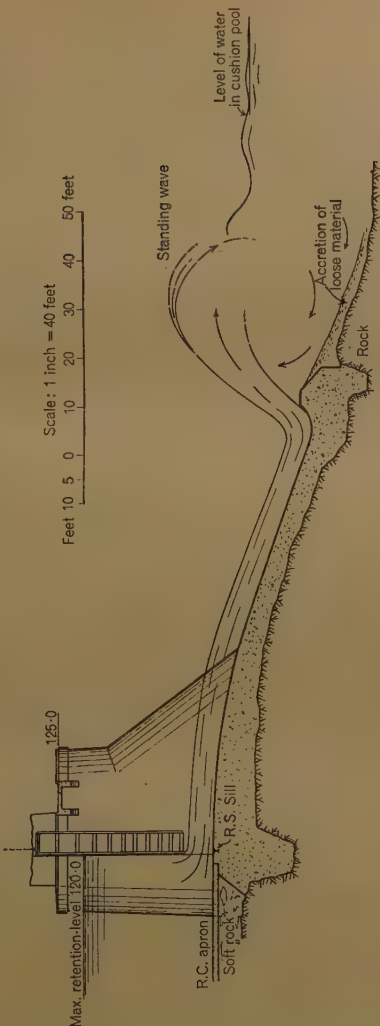
SECTION THROUGH SILL OF GLENLOCHAR BARRAGE.

apron and terminal bucket constructed for the flood-gates at Tongland dam. In action, these flood-gates have passed discharges of 10,000 cusecs, the energy dissipated on these occasions amounting to 25,000 horse-power; after the passage of a flood it has been found that there is actually an accretion of material at the downstream edge of the concrete bucket, whilst the cushioning effect of the relatively deep water in the pool below has prevented appreciable movement of the soft shaley material which forms the bottom.

METHODS OF CONSTRUCTION.

Although the various sections of the works were carried out by different contractors, the methods of construction adopted were generally similar. Owing to limitations of space it is proposed to deal only with the dams and tunnels.

Fig. 25.



SECTION THROUGH SILL AND TERMINAL BUCKET OF TONGLAND FLOOD-GATES.

Construction of Dams.

(a) Excavation.

As a preliminary to the preparation of the contract drawings, the sites of the dams were thoroughly explored by means of trial bores and pits. These investigations showed that conditions in nearly every case were not as favourable as had been expected from the surface examination previously made, and led to certain modifications in the location and design of the works.

In spite of these precautions, it was found on opening up the ground for the foundations of the dams that in many cases the weathering and jointing of the surface rock extended to greater depths than the borings had indicated and necessitated the removal of large quantities of soft and unreliable material in order to obtain reasonable watertightness and an adequate bearing for the foundations. During construction the impermeability of the underlying strata was increased by an extensive programme of pressure-grouting. The river-bed portion of the foundations was constructed in two operations. First, one-half of the river was closed off by a cofferdam within which a block of the main dam was constructed to a little above normal flood-level. This block, which was provided with temporary openings (as may be seen in *Fig. 10*, facing p. 342), on completion permitted the diversion of the river and allowed the second half of the bed to be cofferdammed in its turn.

The type of cofferdam adopted varied with the nature of the river-bed. At Tongland and at Glenlochar, where there was a sufficient depth of gravel overlying the rock, the cofferdams were of single- or double-skin steel sheet-piling, and little difficulty was experienced in making them watertight. In other cases, where rock was at or near the surface, the cofferdams consisted of masonry or concrete walls built during periods of low water and surmounted by timber sheeting.

Normal methods of excavation were used. The bulk of the rock-excavation was done by blasting, for which purpose the most effective explosive proved to be No. 2 Polar Ammon gelignite. Final trimming was usually carried out by barring and wedging. Spoil was loaded into skips or jubilee wagons, hoisted by stiff-legged derricks and run to suitable dumps adjacent to the works. These dumps, if not later to be submerged by the reservoirs, were finally soiled and sown with grass seed.

(*b*) *Grouting.*

In order to avoid excessive loss of cement, the grouting of the rock foundations was not generally commenced until the concrete structure had been carried up to a height of about 20 feet. The general procedure was as follows.

After depositing the first layer of concrete in the bottom of the cut-off trench, 4-inch-diameter steel pipes were placed vertically at intervals of from 5 feet to 10 feet along the line of the trench and drawn up as the successive lifts were poured until there was a sufficient weight of concrete over the foundations to resist possible uplift from the pressure-grouting. Drilling through the cored holes was then commenced with percussion drills, the method of forming the

holes ensuring that the drill broke through the junction between the rock and the concrete. Generally speaking, the holes were arranged alternately as primary and secondary holes, the primary ones being drilled to depths of up to 60 feet below rock-level, while the depth of the secondary holes was usually limited to 10 to 15 feet. After a series of holes had been thoroughly flushed out, grouting of the primary holes was commenced, pressures up to 200 lb. per square inch being applied by means of a reciprocating pump. Pure cement grout with a water/cement ratio (by volume) of 4 to 1 was used at the commencement of operations, the mixture being gradually thickened as injection proceeded, and the process continued until the holes refused to take any more grout at the selected maximum pressure. Sand was never mixed with the grout.

It was commonly found that after a hole had been grouted to refusal, further grout could be forced in 2 or 3 days later. It therefore became the practice to allow a hole to stand after its initial grouting until the injected cement had hardened slightly, and then to flush it out with air and water, re-grouting it again after a lapse of 2 or 3 days. In certain cases it was found necessary to repeat the re-grouting a number of times before the results obtained were considered satisfactory. Before re-grouting a hole, it was tested by filling it with water and applying compressed air at a pressure of 75 lb. per square inch. If the water-level did not fall more than 6 inches in 20 minutes under this pressure, the hole was accepted as being tight and further grouting was not considered necessary.

In certain cases where the rock was of poor quality additional grouting was carried out on the downstream side of the cut-off trench, the work being executed either from rock-level or at a later date from the surface of the concrete block.

Results proved that the maximum grouting pressure of 200 lb. per square inch was not excessive, and examination of the river-bed and banks after the reservoirs were filled indicated that the grouting operations had been quite successful.

(c) *Concreting.*

After completion of excavation the rock surfaces against which concrete was to be placed were cleared of all loose material or shaken rock and then thoroughly cleaned by hosing-down and brushing. In the case of those dams constructed during the early stages of the work a good bond with the rock was secured by covering the surface with a layer of thick cement-grout immediately before the commencement of concreting. This procedure was modified on some of the later dams by the application, in addition to the grout, of a layer of $1 : 1\frac{1}{2}$ mortar $1\frac{1}{2}$ inch thick. Finally, on some of the last dams to

be constructed the grout and mortar were omitted and a bonding layer of specially rich concrete was spread over the rock surface at the commencement of concreting. The same procedure was adopted where fresh concrete was to be deposited on set concrete surfaces, the surface skin of the old concrete being entirely removed before commencing operations. Whilst there is no conclusive evidence with regard to the relative merits of these three methods of bonding to rock or old concrete, the watertightness of the construction-joints in the completed dams would seem to indicate that the use of a covering layer of rich concrete was the most effective. The latter method was also considered to be the most economical, provided reasonable care was taken to avoid wastage of the bonding-layer concrete.

Concrete was spread in layers not more than 2 feet deep, and the size of the block being concreted was restricted to ensure the completion of each layer in 1½ hour. If the deposition of concrete was unavoidably interrupted so that a layer could not be completed in this time, no further work was done until the concrete surface had become sufficiently hard to allow of its being prepared for bonding as described. On most of the dams the height of each lift was limited to 4 feet.

Concreting was discontinued when the temperature of the air or materials dropped to 38° F., and the minimum temperature at which it was recommended with a rising thermometer was 34° F. On three sections of the works concreting was made possible during temperatures below those specified by steam-heating the materials and mixing-water. This did not prove very satisfactory, and was not generally permitted since, under certain conditions, it was difficult to ensure complete heating of the materials and, in some cases, to avoid overheating them.

The shear strength of the horizontal joints was improved either by providing grooves in the top of the completed concrete lift, or more generally by finishing off the top in a series of steps rising towards the downstream side of the dam. In some cases, as an additional measure of precaution, bonding stones were left projecting above the top of each lift.

The dams were constructed in bays about 50 feet long and to the full width of the structure. These bays were separated by closing spaces about 5 feet wide (*Fig. 10*, facing p. 342) which were not filled until the concrete in the adjoining bays had aged sufficiently for most of the shrinkage to have taken place.

In the closing-space faces of the main bays vertical keys were formed, and in the case of the arch dams provision was also made for grouting the joints under pressure after the gaps had been filled

with concrete. For this purpose pre-cast porous concrete pipes were built up vertically in the upstream keys as concreting of the closing space proceeded. These porous pipes were in lengths of 2 or 4 feet and were approximately 9 inches square with a hole about 3 inches in diameter cored through the middle, the continuity of the pipes being broken every 30 feet or thereabouts by a 2-foot layer of concrete. From the lower end of each 30-foot section of porous pipes a 1-inch-diameter grouting pipe was laid horizontally to the downstream face of the dam, whilst a $\frac{1}{2}$-inch-diameter escape-pipe was similarly provided from the top end of each section. Grouting operations were delayed as long as possible to allow for maximum shrinkage in the concrete. In grouting a section of porous pipe, the escape-pipe was plugged when grout commenced to flow through it and injection was then completed at a pressure of about 50 lb. per square inch.

Construction of Tunnels.

(*a*) *Surveys.*

In the case of the Glenlee tunnel, the Contractor's survey was based on a system of triangulation. To check the setting-out the Engineers established the tunnel centre-line on the ground surface.

In transferring the tunnel centre-line from the ground surface to the bottom of the Craigshinnie shaft, two plumb-wires were used carrying weights immersed in oil and providing a base-line 8 feet long. The Contractor carried his line through the heading by means of the Weisbach triangle method, whilst the Engineers adopted the co-planar method as a check.

When carrying the centre-line through the heading, a small graduated steel scale was fixed to each of the pegs in the crown of the tunnel at right angles to the line of the tunnel. In extending the line forward by theodolite, the scale-reading at each peg was observed and noted for reference; this method proved to be of great assistance when comparing a number of different check lines for the purpose of establishing a more accurate average line.

(*b*) *Excavation.*

The Glenlee tunnel, which has a total length of 18,988 feet, runs entirely through greywacke. Excavation was carried on from six faces, one from the Glenlee portal, two from an adit at chainage 3,600, two from the Craigshinnie shaft at chainage 9,560, and one from the intake at Clatteringshaws. The adit was located at a point where a horizontal heading could be conveniently driven and

where reasonably level ground was available for the establishment of compressors and other temporary plant. The location of the Craigshinnie shaft was selected with the object of dividing the length of tunnel between the intake and adit into two approximately equal sections and at the same time avoiding the use of an unnecessarily deep shaft. After the completion of the works this shaft was used to bring additional water into the tunnel from two adjacent burns.

In five out of the six headings driving was carried out by the full-face method, but in the case of the heading driven from the intake-end heading and benching were used with the object of reducing trouble with water on a down grade and also of obtaining a larger size of stone for use in pitching the intake-embankment.

In general, drilling was done at each face by four drifters mounted on two columns. Firing was carried out electrically with delayed-action detonators, and here again Nobel's No. 2 Polar Ammon gelignite was found to be the most effective explosive. The average length of pull was about 6 feet, and the average quantity of gelignite used per cubic yard of rock excavated (based on the designed cross-section) was 3½ lb. During the earlier stages of driving the Contractor drilled the outer ring of holes well inside the pay line, and although this method reduced overbreak to a very low figure, it led to a considerable amount of trimming. Later, in order to avoid holding-up the concrete work, wider drilling was resorted to in spite of the additional overbreak which this occasioned.

Considering the tunnel as a whole, the rock conditions were very favourable, and, apart from a short length near the portal and a section of approximately 1,000 feet where the tunnel passed beneath Craigshinnie Burn, the rock was self-supporting. Where support was necessary, steel ribs formed to a circular shape from 5-inch by 3-inch rolled steel joists were used at 5-foot centres in conjunction with heavy corrugated steel lagging backed by brickwork. These ribs were subsequently built into the permanent concrete lining of the tunnel.

During the early stages of driving spoil was loaded into the wagons by hand, but later Sullivan pneumatically-driven scrapers were adopted and gave very good service and much faster loading.

Electric battery locomotives were used for handling the spoil-trains during the driving period, but after the main excavation had been completed and a through ventilation of the tunnel could be obtained the engineers agreed to the use of diesel locomotives. After excavation was completed one-half of the Craigshinnie shaft was divided off by a timber bulkhead and used as a storage-bin for sand and stone, the materials being drawn off through hoppers at the bottom.

The average time taken for the different driving operations were as follows :—

Blowing out fumes after firing and clearing tracks . .	1½ hours
Removing spoil with Sullivan scraper	2 ,,
Setting up columns and drills	1½ ,,
Drilling	2¼ ,,
Charging holes and blasting	½ ,,
Total	8 ,,

Forced ventilation of the working faces was employed, the specification requiring a minimum of 4,500 cubic feet of free air per hour per man. The system was so arranged that air could be either blown into or drawn from the tunnel, the latter method generally being adopted. The exhausts of the diesel locomotives which were used during the latter stages of the work were fitted with special cleaning boxes, which proved effective in preventing the discharge of harmful gases.

Electric power was supplied at 11,000 volts from the Central Electricity Board's temporary sub-station at Glenlee, a single-circuit line on wooden poles being taken along the line of the tunnel, and supplies being tapped off where required and stepped down to 400 volts for driving the Contractor's plant. The Contractor was required to provide an independent stand-by power-station, and to meet this condition two 100-kilowatt diesel-driven generating sets were installed.

The following figures relative to the excavation of the Glenlee tunnel are of interest :—

Explosives used per round	90 to 110 lb., depending upon quality of rock
Best week's progress at one heading	136 feet
Average week's progress for whole tunnel . .	100 feet per heading

At the Carsphairn Lane end of the Doon tunnel, where the ground consisted of water-bearing moraine, it was necessary to excavate a length of about 200 feet under compressed air. When the bad ground was reached an air-lock was built across the heading and the excavation carried on under an air-pressure of 14 lb. per square inch. To support the completed excavation a brick lining was used, the lining being kept as close as possible to the working face as excavation proceeded.

The arrangements adopted for ventilation were similar to those already described for the Glenlee tunnel.

The following figures relate to the progress attained :—

		Doon Tunnel.	*Deugh Tunnel.*
Average monthly driving progress	(one face)	375 feet	452 feet
Best month's progress	(,,)	617 feet	607 feet

In order to reduce trimming to a minimum, the Contractor elected to drive a relatively larger heading than was adopted for the Glenlee tunnel, the average overbreak resulting being shown by the following figures :—

	Doon Tunnel.	*Deugh Tunnel.*
Excess of excavation over specified minimum average section . . .	29 per cent.	26 per cent.
Excess of concrete lining over specified minimum average thickness . .	127 per cent.	108 per cent.

The quantity of explosive used was about 4½ lb. per cubic yard of rock, based on the designed cross section of the tunnel-excavations, or 3½ lb. per cubic yard if based upon the quantity of rock actually removed.

Tunnel-Concreting.

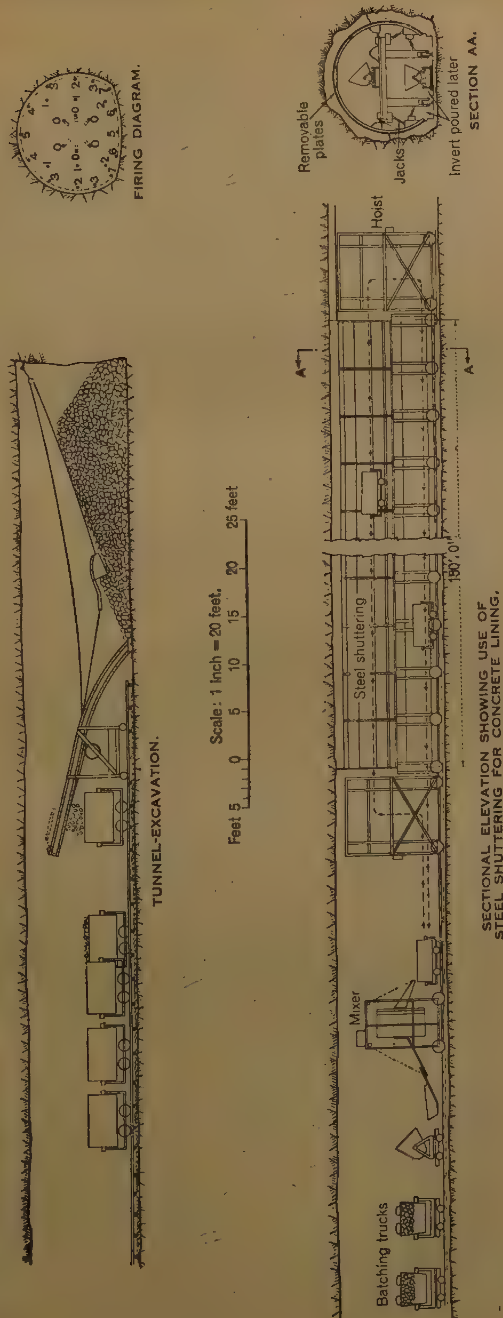
The unusually long shuttering used for the lining of the Glenlee tunnel was one of the most interesting features of the work. The concreting gantry (*Figs. 26*), which was entirely of steel, was of the collapsible, self-supporting type and consisted of a single unit, 150 feet in length, which permitted the walls and crown to be poured in one continuous operation. The gantry was articulated so that it could negotiate the several large-radius bends which occurred in the tunnel.

In carrying out the lining operations, the first step consisted in screeding a narrow strip of the invert at the base of the wall on either side of the tunnel, after which the rails carrying the gantry were mounted upon concrete pads and accurately set. The gantry was then run forward, accurately lined-up, and the segmental steel panels which formed the shuttering expanded by jacks until their lower edges nipped the strip at the base of the walls.

Contrary to the more general practice, the concrete-materials after being batched outside the tunnel were transported to a mixer located at one end of the form, where, after mixing, the concrete was fed into special side-tipping wagons. These wagons were then hoisted on to the raised platform which ran the whole length of the form, and were pushed to the point where concrete was being deposited. The empty wagons were afterwards lowered at the far end of the gantry and were run back to the mixer along the tunnel floor. All placing of concrete was done by hand, and during the process the shuttering was vibrated by pneumatic hammers.

Once the men had become accustomed to the work it was found possible to complete the filling of the whole 150-foot length of shuttering in 24 hours. After the lapse of a further 18 hours the

Figs. 26.



EXCAVATION AND CONCRETING OF GLENLEE TUNNEL.

shuttering was eased away from the face by means of jacks and the whole unit run forward on rails to its next position.

Whilst the great length of the shuttering used permitted a high rate of progress to be attained, it was not easy, owing to the confined space in which the men had to work, to ensure that the concreting was carried out smoothly and continuously, and rough joints in the concrete lining were not infrequent. As a result of their experience at Glenlee the Authors would hesitate to advise the adoption of such an excessively long gantry for a tunnel of similar size.

The best month's progress was 2,250 feet, whilst the average for the whole tunnel was 1,300 feet.

After the concrete lining was completed holes were drilled through the crown of the tunnel at 10-foot centres and grout was injected at a maximum pressure of 50 lb. per square inch. The average amount of cement injected behind the crown and elsewhere where necessary to seal leaks was 1·55 cwt. per linear foot of tunnel. The grout used had a water/cement ratio (by volume) of 1 to 1. After the completion of the grouting operations the total leakage into the tunnel did not exceed 0·06 cusec over the whole length of nearly 19,000 feet.

In the Doon–Deugh tunnels the concreting of the lining was carried out simultaneously with, and at a convenient distance behind, the excavation of the headings. The Deugh tunnel, which was designed to be free-flowing, was, except at those places where the roof needed support, lined only up to the springing of the crown; but the Doon tunnel, which was intended to work under pressure, was lined all round, and the crown afterwards grouted. Travelling shutters 30 feet long were used for the walls and crown, which were concreted separately, the invert being placed last without the use of shuttering.

Where the Doon tunnel ran through the bad ground already referred to, a heavily-reinforced concrete lining 4 inches thick designed to withstand the whole bursting pressure was gunned on to the brickwork. Apart from this, the concrete work in the Doon–Deugh tunnels was quite straightforward and normal methods of construction were adopted.

Figures relating to the concreting of the Doon–Deugh tunnel linings are given below :—

	Doon Tunnel.	*Deugh Tunnel.*
Best month's progress	882 feet	795 feet
Average monthly progress . . .	432 feet	376 feet

Control of Concrete.

Aggregate.

Generally speaking, the coarse aggregates used for concrete were obtained either from quarries opened up close to the site of the works, or from the excavations themselves. The principal exception occurred in the case of the Tongland aqueduct, the $\frac{5}{8}$-inch granite for which was obtained from the quarries at Dalbeattie.

The greywacke, a rock of sedimentary origin, which provided most of the coarse aggregate for the works, broke into flaky pieces when crushed in ordinary jaw crushers, and it was found that a predominance of these flaky pieces in the coarse aggregate had a more harmful effect on the workability of the concrete than the use of a poorly-graded material. For the Glenlee tunnel and the Earlstoun and Carsfad works, impact breakers were used instead of jaw crushers for breaking the stone. These produced less flaky stone, and led to a marked improvement in the workability of the concrete, which approached that obtained by using gravel for a coarse aggregate. In the Authors' opinion too little attention is usually given to the shape of the individual particles when considering the suitability of a particular aggregate, and, whilst the importance of grading is generally fully appreciated, it frequently happens that the quality of a concrete could be improved by the use of a more suitable crushing plant.

Sand.

Considerable difficulty was experienced in obtaining sand of a suitable quality. On some sections of the work a mixture of pit-sand and granite or greywacke screenings was used as a fine aggregate, equal proportions being usually adopted. Screenings containing more than 6 per cent. of dust were rejected. Typical results of tests carried out in the works laboratory (Table VI) demonstrate the detrimental effect of the dust which is usually found in the screenings from crushed stone.

Table VI.

Dust (finer than No. 100 sieve) in fine aggregate, greywacke screenings : per cent. . .	Nil	5	10	15	20
Mean strength of three 3 : 1 mortar briquettes at 7 days : lb. per square inch . . .	500	497	462	413	357

It was also found that adding too large a proportion of screenings

to the pit-sand had an adverse effect on the strength of the resulting concrete, as is shown by Table VII.

TABLE VII.—COMPRESSION-TESTS ON CONCRETE SPECIMENS.

Mix.	Stone.	Fine aggregate: Pit-sand: per cent.	Fine aggregate: Screenings: per cent.	Strength at 7 days: lb. per square inch.
1 : 2 : 3	¾-inch greywacke	100	—	2,284
,,	,,	75	25	2,362
,,	,,	50	50	2,204
,,	,,	25	75	1,882
,,	,,	—	100	1,614

As a result of the peaty character of the surrounding country, considerable trouble was met on account of the presence of vegetable matter in the supplies of pit sand. Sand failing to pass a specified sodium-hydroxide colour test was rejected, as was also sand containing more than 4 per cent. of loam. Although it is recognized that some types of vegetable matter may not be harmful, that met with in Galloway had a markedly deleterious effect on the strength of the concrete, and in most cases where the concrete showed an exceptional falling-off in strength, the cause was traced to the presence of an excessive amount of vegetable matter.

Cement.

All consignments of cement intended for use on the works were tested at the manufacturer's works and were not accepted for delivery until the results of the 7-day tests were known to be satisfactory. After delivery to the site the cement was again tested, and these tests were usually repeated if a particular consignment had been on the site for more than a month.

Considerable trouble was experienced at one period on account of the fast-setting properties of certain brands of cement, which produced excessive shrinkage and cracking of the concrete. To reduce trouble from this cause, arrangements were made with the manufacturers concerned to supply cement of a coarser grinding. It was arranged that a minimum of 6·5 per cent. of the cement should be retained on a No. 170 sieve, and this modification in the Specification proved quite effective in reducing the development of shrinkage-cracks. For the same reason the use of special rapid-hardening cement had previously been discontinued.

Proportions of Concrete.

The principal concrete-mixtures used on the works are set out in Table VIII below :—

TABLE VIII.

Class.	Specified proportions.			Approximate mix.	Size of stone : inches.	Specified minimum compressive strength at 7 days : lb. per square inch.	
	Cement : cwt.	Sand : cubic feet.	Stone : cubic feet.			Ordinary Portland cement.	Rapid-hardening cement.
" O "	3	12	20	1 : 3 : 5	2	1,200	1,900
" D "	4	10	20	1 : 2 : 4	1	—	2,400
" S "	5	12	20	1 : 2 : 3	1	1,700	—
" M "	$6\frac{1}{2}$	12	20	1 : $1\frac{1}{2}$: $2\frac{1}{2}$	1	2,000	3,000
" R "	10	12	20	1 : 1 : $1\frac{1}{2}$	$\frac{5}{8}$	2,400	3,600

In certain cases variations were introduced into the specified proportions of the concrete where it was found that by so doing the workability of the mix could be improved without affecting its strength.

Class " O " concrete was used in the hearting of dams, wing-walls and retaining walls, foundations for power-stations and continuous saddles for aqueducts, anchorages and piers for pipe-lines, and in mass-concrete work generally.

Class " D " concrete was used in the Glenlee tunnel-lining.

Class " S " concrete was used in the Doon–Deugh tunnel-linings, canal-linings, intake-towers and in power-station walls, floors, and roofs.

Class " M " concrete was used in the upstream facing of dams and spillways, cut-off walls, base layers of dams, crests of dams and spillways, and around draw-off pipes and temporary openings through the dams.

Class " R " concrete was used for the reinforced-concrete pressure-aqueducts.

Mixing of Concrete.

Practically all concrete was machine-mixed. The materials were mixed for a minimum time of $1\frac{1}{2}$ minute after the whole batch, including water, had entered the mixing drum. Weigh-batching plant was extensively used on the works.

Control of Water.

In the absence of facilities for inundating the aggregates, the

quantity of water used was controlled by frequent slump-tests, the maximum slumps allowed being as follows :—

For mass concrete	Between 1 inch and 2 inches
For lightly-reinforced concrete work .	Not more than 3 inches
For heavily-reinforced concrete work .	Not more than 4 inches

Tests on Concrete.

Specimens for crushing tests were prepared by filling 6-inch-cube cast-iron moulds from a representative sample of freshly-deposited concrete. When sufficiently hard the cubes were stripped from their moulds and cured in damp sand thermostatically maintained at a constant temperature of 60° F. They were generally broken when 7 days old, but periodically additional cubes were made for testing at 28 days and 3 months. On the Stage I works, four concrete specimens were prepared for each test. Experience proved, however, that this number could be reduced without affecting the reliability of the results, and on the Stage II works only two specimens for each test were prepared. On each occasion when test cubes were made, samples were taken of the cement, sand, and stone in use at the time, for further investigation in case the results of the cube tests should be unsatisfactory.

On those occasions when there was reason to doubt the quality of any particular lot of deposited concrete, test-blocks were cut from the suspected portion of the structure and the usual compression-tests were carried out. In some cases the samples were chemically analysed.

Periodic tests were carried out to determine the porosity of concrete, these tests being generally confined to those mixes which were used in situations where impermeability was important.

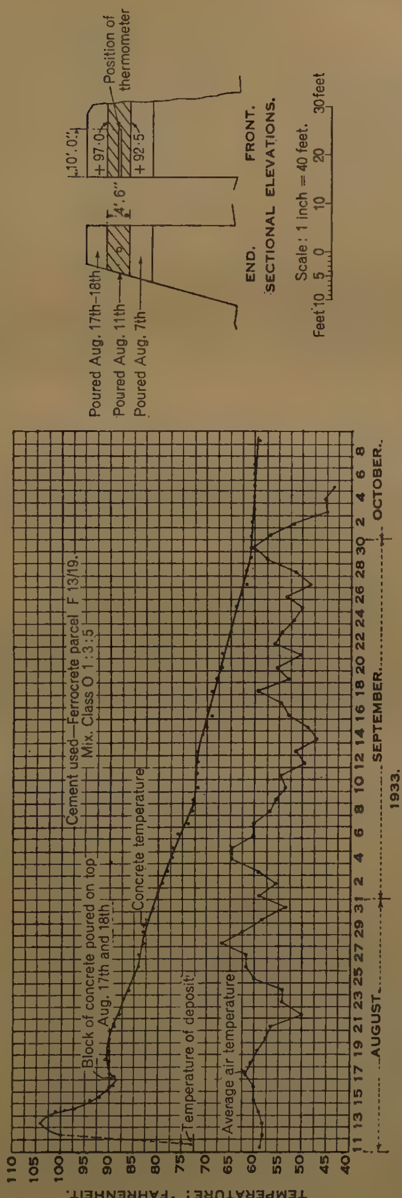
On the principal dams, records were kept of the internal temperatures of the structures during maturing of the concrete. The results of one of these tests are given in *Figs. 27*.

In Table IX (p. 370) are given the average crushing strengths of the specimens made from the principal classes of deposited concrete. It will be seen that the average strength of the concrete used was more than 1½ times the specified minimum.

General Procedure Adopted for Concrete Tests.

Tests on cement, compression and porosity tests on cement specimens, and special tests, such as analyses of set concrete, were undertaken by Mr. R. H. H. Stanger, Assoc. M. Inst. C.E., and a member of Mr. Stanger's testing staff was resident on the works. A site laboratory, with the necessary apparatus for carrying out

Figs. 27.



Temperature-Changes in Concrete of Tongland Dam.

TABLE IX.—MEAN STRENGTH OF CONCRETES USED ON WORKS.

Class.	Approximate mix.	Number of tests made.	Average crushing strength at 7 days : lb. per square inch.	Percentage of specified strength.	Percentage of tests below specified strength.
"O" (ordinary)	1 : 3 : 5	203	1,960	164	9
"O" (rapid-hardening) .	1 : 3 : 5	45	2,500	132	18
"D" (rapid-hardening) .	1 : 2 : 4	85	3,750	156	7
"S" (ordinary)	1 : 2 : 3	97	2,665	157	1
"S" (rapid-hardening) .	1 : 2 : 3	21	3,560	142	10
"M" (ordinary)	1 : 1½ : 2½	119	3,670	184	6
"M" (rapid-hardening)	1 : 1½ : 2½	17	4,250	142	12
"R" (ordinary)	1 : 1 : 1½	39	4,734	198	Nil
"R" (rapid-hardening)	1 : 1 : 1½	57	4,850	135	2

tests on concrete materials, mortar-strength tests, etc., was established adjacent to the Engineer's Office.

COST OF WORKS.

The total expenditure on the Galloway hydro-electric development amounted approximately to £3,000,000, of which £1,840,000 was accounted for by the constructional engineering contracts. The individual costs of the principal works are set out in Appendix I (p. 372), and unit costs of construction in Appendix II (p. 373).

CONCLUSION.

The construction of the works was undertaken in two stages, and occupied a period of 5 years, being completed in the autumn of 1936.

The constructional works were designed and carried out under the direction of Sir Alexander Gibb & Partners, who acted throughout in co-operation with Messrs. Merz & McLellan, the Consulting Engineers for the electrical portion of the scheme. The responsibility for the design and supervision of the works devolved upon Mr. James Williamson, M. Inst. C.E., who until recently was Chief Engineer for Sir Alexander Gibb & Partners, whilst Mr. William Hudson, B.Sc. (Eng.), M. Inst. C.E., acted as Superintending Civil Engineer throughout the whole period of construction.

From time to time the following acted as Resident Engineers on the different sections of the works :—

Mr. A. R. Adams, B.Sc. (Eng.), Assoc. M. Inst. C.E.	Clatteringshaws dam and Doon–Deugh tunnels.
Mr. R. G. Edkins, B.A., Assoc. M. Inst. C.E.	Clatteringshaws dam.
Mr. S. S. Harrison, Assoc. M. Inst. C.E. . .	Kendoon works.
Mr. J. K. Hunter, B.Sc. (Eng.), M. Inst. C.E.	Tongland works.
Mr. C. C. Marshall, B.Sc. (Eng.), Assoc. M. Inst. C.E.	Loch Doon dam.
Mr. T. A. L. Paton, B.Sc. (Eng.), Assoc. M. Inst. C.E.	Glenlee tunnel.
Mr. Guy Richards, B.A., Assoc. M. Inst. C.E.	Glenlee power-station and pipe-line, Carsfad works.
Mr. M. A. Speir, Assoc. M. Inst. C.E. . . .	Glenlee tunnel and Earlstoun works.
Mr. B. W. Tawse, M.A., Assoc. M. Inst. C.E.	Tongland and Kendoon works.

Mr. Walter Weed was chief draughtsman throughout the greater portion of the period occupied by the design of the works, and to him the Authors wish to express their thanks for the assistance he has rendered in the preparation of the Paper.

The Authors' thanks are also due to the Galloway Water Power Company and the Central Electricity Board for permission to present this Paper, and to Sir Alexander Gibb & Partners for the facilities given for its preparation.

A list of the principal Contractors who were responsible for the execution of the constructional work is given in Appendix III (p. 375).

The Paper is accompanied by thirty-six sheets of drawings and six photographs, from some of which Plates 1 and 2, the Figures in the text, and the half-tone page-plate have been prepared, and by four Appendixes (three of which are printed).

[APPENDIXES.

APPENDIX I.

Cost of Principal Works.

	£	£
Glenlee Works.		
Clatteringshaws dam	125,500	
Pullaugh Burn aqueduct	12,750	
Road-diversions and bridges	25,000	
Tunnel-intake works	20,500	
Craigshinnie shaft and tunnel-adit	6,600	
Surge-shaft	7,500	
Glenlee tunnel	211,000	
Pipe-line, including foundations	23,000	
Portal-valve, guard-valves, valve-house, etc.	10,250	
Power-house	31,500	
Access-roads	2,250	
Tailrace and tailrace bridges	16,400	
Miscellaneous works	3,500	
		496,000
Tongland Works.		
Glenlochar barrage	25,500	
Tongland dam and spillway-works	136,000	
Fish-pass	11,000	
Intake-works, screens and gate	14,500	
Tunnel	14,000	
Reinforced-concrete aqueduct	61,000	
Surge-tower, overflow-pipe, trifurcation and turbine-pipes	28,000	
Valve-house foundations and superstructure	5,000	
Power-house	62,500	
Tailrace work	8,000	
Road-diversions	5,500	
Miscellaneous works	4,000	
		375,000
Loch Doon Reservoir.		
Dams	80,000	
Fish-pass	6,000	
Roads and bridges	23,000	
		109,000
Doon–Deugh Tunnel Works.		
Loch Doon tunnel	52,000	
Loch Doon intake	9,000	
Deugh tunnel	52,000	
Aqueducts, intakes, etc.	44,000	
Roads and bridges	14,000	
		171,000
Carry forward		1,151,000

	£	£
Brought forward		1,151,000
Kendoon Works.		
Dams	168,000	
Canals and aqueduct	86,000	
Surge-tower	6,250	
Pipe-line	26,000	
Valves and valve-house	11,000	
Power-station	27,500	
Roads and bridges	16,250	
		341,000
Carsfad Works.		
Dam	112,000	
Intake and pipes	22,500	
Power-station	23,500	
Fish-ladder	6,000	
Roads	4,500	
		168,500
Earlstoun Works.		
Dam	69,000	
Canal, intake and pipes	53,500	
Power-station	24,500	
Fish-ladder	8,500	
Roads and bridges	17,500	
		173,000
Miscellaneous works		6,500
Total cost of constructional work		£1,840,000

APPENDIX II.

Unit Costs of Construction: Average Rates.

Dams.	*Unit.*	*Rate.* £	s.	d.
Trench excavation in soft material in river-bed	cubic yard		9	9
Trench excavation in rock in river-bed	,,		11	9
Trench excavation in soft material in banks	,,		5	0
Trench excavation in rock in banks	,,		12	3
Excavation in soft material for spillways, canals, etc.	,,		2	3
Excavation in rock for spillways and canals	,,		7	6
Concrete, class "O," in hearting and mass work generally	,,	1	4	6
Concrete, class "M," in facing and piers, footways, etc.	,,	1	18	0
Concrete, class "S"	,,	2	4	0

	Unit.	Rate. £	s.	d.
Reinforced-Concrete Pressure Aqueducts.				
Concrete, class "R," in reinforced shell	cubic yard	3	1	6
Concrete, class "O," in saddle	,,	1	13	6
Steel reinforcement of shell	ton	12	4	0
Power-Stations.				
Concrete, class "O," in mass foundations	cubic yard	1	13	0
Concrete, class "D," in foundations	,,	1	19	0
Concrete, class "M," ,,	,,	2	2	0
Concrete, class "S," ,,	,,	2	4	0
Concrete, class "S," in walls, floors, etc.	,,	3	2	6
Steel reinforcement	ton	12	2	6
Structural steelwork	,,	15	0	0
Surge-Tanks.				
Steel plates	,,	16	7	6
Pipe-Lines.				
Riveted steel, straight pipes	,,	20	15	0
Welded steel, ,,	,,	19	11	0
Bifurcations	,,	28	15	0
Trifurcation (Tongland)	,,	25	10	0
Tunnels.				
Glenlee, excavation and concrete lining	linear foot	9	0	0*
Doon ,, ,, ,,	,,	5	16	0*
Deugh ,, ,, ,,	,,	5	3	0*
Cementation of Dam-Foundations.				
Drilling through rock or concrete 0–20 feet	,,		4	1
,, ,, ,, 20–30 ,,	,,		7	2
,, ,, ,, 30–40 ,,	,,		10	2
,, ,, ,, 40–50 ,,	,,	1	0	2
,, ,, ,, 50–60 ,,	,,		14	0†
,, ,, ,, 60–70 ,,	,,		18	0†
,, ,, ,, 70–80 ,,	,,	1	2	7†
Cement used for cementation	ton	8	2	0

NOTE.—In all cases the rates for concrete include the cost of shuttering. All steelwork prices include erection.

* These prices are for plain lined tunnel only, and exclude special or reinforced sections, grouting, etc.

† These drilling rates were applicable to one dam only.

APPENDIX III.

Principal Contractors for Constructional Work.

Sluices for Glenlochar barrage Gates for Tongland dam	Glenfield & Kennedy, Ltd.
Glenlochar barrage—foundations and piers Tongland dam and tunnel	John Howard & Co., Ltd.
Road-diversions at Clatteringshaws and Tongland Glenlee tunnel Glenlee power-station and tailrace Earlstoun and Carsfad dams and power-stations Kendoon power-station Kendoon aqueduct	A. M. Carmichael.
Clatteringshaws dam	Shanks & McEwan, Ltd.
Tongland aqueduct Tongland power-station	W. Taylor & Son (Glasgow), Ltd.
Doon–Deugh tunnels	Charles Brand & Son, Ltd.
Kendoon reservoir-works Loch Doon dam	Sir Robert McAlpine & Sons (Newcastle-upon Tyne), Ltd.
Glenlee pipe-line Kendoon pipe-line and surge-tower	Sir William Arrol & Co., Ltd.
Kendoon valves	Boving & Co., Ltd.
Road-diversions at loch Doon and Kendoon	W. Binnie & Sons, Ltd.

Paper No. 5163.

"The Galloway Hydro-Electric Development, with Special Reference to the Mechanical and Electrical Plant." †

By William Hawthorne, B.E., M. Inst. C.E., and Frederick Herbert Williams, B.Sc. Tech., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The main function of the Galloway Water Power Company's development is to supply power during times of peak load to the industrial areas of Central Scotland and North-West England. The district in which the development is situated lies roughly mid-way between these areas (*Fig. 28*), and the power is transmitted northwards and southwards, as desired, over the National Grid line which runs from Carlisle to Kilmarnock and forms the western Grid interconnexion between England and Scotland.

This Paper deals with the purpose of the scheme, some features of the plant, and the results obtained in operation. Descriptions are given elsewhere [1] of the special features incorporated in the hydraulic

† Correspondence on this Paper can be accepted until the 15th July, 1938.—Sec. Inst. C.E.

[1] W. Hudson and J. K. Hunter, "The Galloway Hydro-Electric Development, with Special Reference to the Constructional Works," p. 323 (*ante*). R. W. Mountain, "The Galloway Hydro-Electric Development, with Special Reference to its Interconnexion with the Grid," p. 407 (*post*).

works of the scheme, and of the installations of the Central Electricity Board associated with it. Some notes on the hydraulic plant have previously been published.[1]

Purpose of the Scheme.

The scheme is the first important instance in Great Britain of plant being installed to deal primarily with the daily peak which is found in the load-curves of most supply-undertakings, and which may

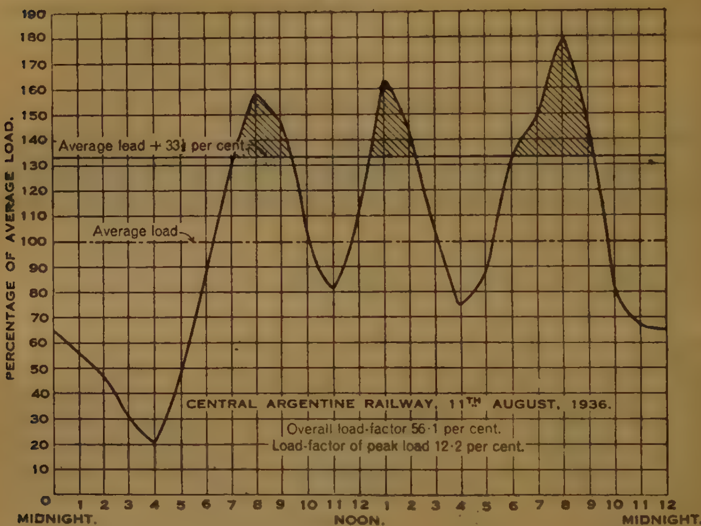
Fig. 28.

Map of Galloway Area.

be expected to increase in magnitude and sharpness as the domestic use of electricity develops. *Figs. 29* (pp. 378, 379) illustrate the way in which the peak-load problem varies in different undertakings; *Fig. 29* (*a*) represents the load of a power-station supplying a suburban railway, *Fig. 29* (*b*) the load of a municipality, and *Fig. 29* (*c*) the combined load-curve of the British Grid. The term "peak" may

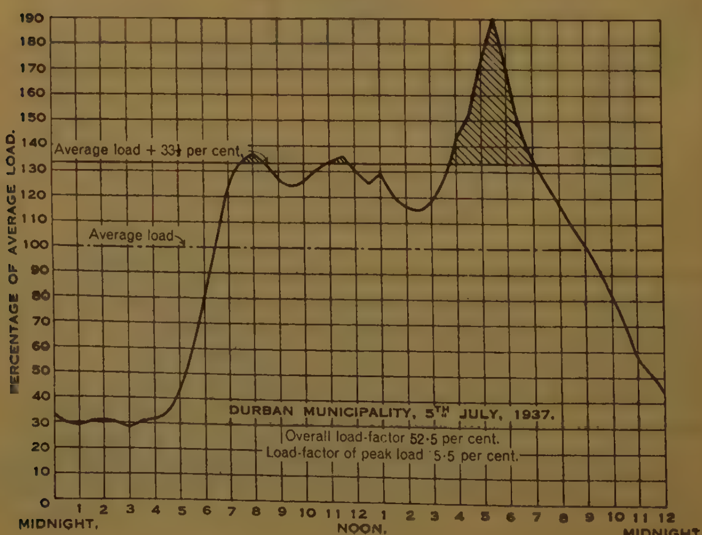
[1] Dr. P. W. Seewer, "Recent Developments in Hydro-Electric Engineering, with special reference to British Practice." Proc. I. Mech. E., vol. 134 (1937), p. 283.

Fig. 29 (a).



be variously interpreted ; in these diagrams the peaks are shown as the loads in excess of four-thirds of the average load ; they amount to 4,000, 17,230 and 944,000 kilowatts respectively. On this defini-

Fig. 29 (b).

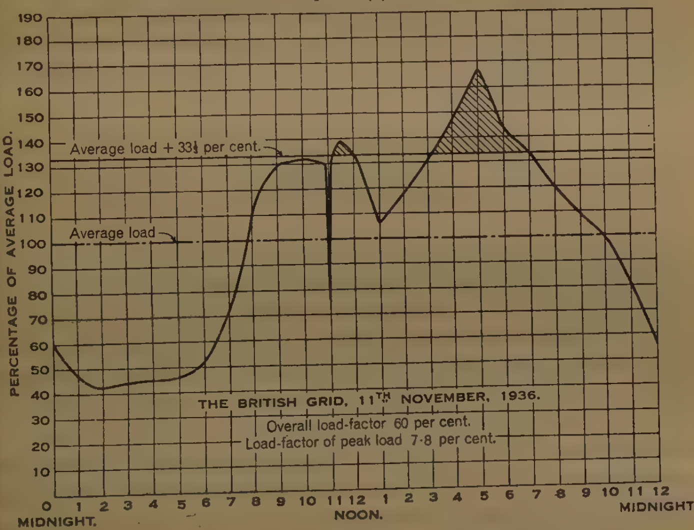


tion the load-factors in these three examples for the particular twenty-four hours illustrated are :—

	Railway.	*Municipality.*	*Grid.*
Overall load-factor, based on maximum demand and total units : per cent. . .	56·1	52·5	60
Load-factor, excluding peak-load and peak-plant : per cent.	71	72·6	73
Load-factor of peak-plant : per cent. . .	12·2	5·5	7·8

It can be calculated from these figures that if the capital charges per kilowatt and operating costs per unit are the same for the peak-load plant as for the rest of the installation, the cost per kilowatt-

Fig. 29 (c).



hour of peak energy may be from 3 to 5 times that of the energy produced during off-peak hours. In a station mainly used for supplying peak-load power, therefore, it is even more desirable than in a base-load station that the capital charges, operating costs, or both, should be low. Apart from establishing new highly-efficient base-load stations and using as peak-load plant the least efficient of the older installations (on which the capital expenditure has been wholly or partly redeemed by the operation of a sinking fund, and which in addition often have the advantage of being near the centre of the load), three possible ways of producing cheap peak-hour units are :—

(1) To erect near the centre of load steam- or oil-engined stations in which the capital cost is kept low, even at the expense of some reduction of efficiency and life.

(2) To instal "pumped storage" plants in which water is pumped into elevated reservoirs during hours when the demand on other connected plants is light, and is withdrawn during peak hours through turbines which drive generators feeding into the main network.

(3) To develop hydro-electric schemes in which the natural run-off is stored during hours of light load and utilized during the peak hours.

The Galloway development is an example of the last-named way of dealing with the peak-load problem; it is specially suitable for the purpose, since the constructional works were relatively inexpensive and the operating charges are particularly low.

The extent to which the Galloway development relieves the peak demand on the Grid is seen in Fig. 30, Plate 3, which shows the maximum daily output from the system.

Supply of Water.

In the Galloway catchment-area, two-thirds of the rain falls during the six winter months, and consequently the ability of the plant to turn out units is greater during these months than in summer. This fits in well with the general power-requirements of the country, which are much less in summer than in winter. Reliance is not placed entirely on daily rainfall and run-off, for, in addition to the head-ponds above the power-stations which provide the daily storage, two reservoirs have been created for seasonal storage. One of these was an existing lake, loch Doon, part of the outflow from which is diverted from its natural northward course and passed southwards (via the rivers Deugh and Ken and their continuation the Dee) through power-stations at Kendoon, Carsfad, Earlstoun and Tongland. This loch contains storage equivalent to 18,000,000 kilowatt-hours. The other seasonal reservoir is the artificial Clatteringshaws loch, formed by damming the valley of the Blackwater of Dee; the water from it passes through a tunnel to Glenlee power-station whence it flows into the river Ken below Earlstoun and is used again at Tongland. A complete draw-down of this loch (without any inflow) would produce 12,000,000 kilowatt-hours. Loch Ken, across the lower end of which, at Glenlochar, a barrage has been constructed, may be regarded as partly seasonal and partly daily storage. It can provide 650,000 kilowatt-hours.

The station head-ponds are of small capacity, and the number of

units they can generate without being drawn down to an uneconomic level is strictly limited. The limitations are as follows :—

Kendoon.—The head-pond feeding this station has a normal draw-down of 8 feet, which gives 5–6 hours' full-load running with no inflow. The reservoir is replenished in normal times from loch Doon through the valve at Drumjohn ; the full discharge of this valve when the level of loch Doon is high is equivalent roughly to one-quarter of the full load of the station. It takes from 5 to 7 hours for the water released at Drumjohn to reach Kendoon power-station intake at Blackwater. When this head-pond is fully drawn down the water will produce over 230,000 kilowatt-hours in Kendoon, Carsfad and Earlstoun power-stations.

Carsfad.—The head-pond for this station is fed direct from the tailrace of Kendoon power-station. About ½ to ¾ hour (depending on load) elapses between the times water is released at Kendoon and becomes available at Carsfad. The economical draw-down is 2 feet, which corresponds to about 1 hour's full-load running of the station assuming no inflow. The normal draw-down is 8 feet, which allows of full-load running for about 4½ hours without inflow and produces 55,000 kilowatt-hours.

Earlstoun.—The head-pond is fed direct from Carsfad power-station, the time-lag between the release of the water at Carsfad and its arrival at Earlstoun being 30 minutes as a minimum. The economical draw-down is again 2 feet, but owing to greater storage-capacity this allows of full-load running for about 1¼ hour without inflow. The normal draw-down is again 8 feet, corresponding to full-load running for about 5 hours without replenishment and an output of 60,000 kilowatt-hours.

Tongland.—The head-pond for this station has a normal draw-down of 10 feet, which corresponds to about 2 hours' full-load running of the station and an output of 65,000 kilowatt-hours. Thereafter the running depends, in normal weather, on the inflow, which, owing to restrictions imposed by conditions outside the Company's control, must not, except in flood, be allowed to exceed the flow corresponding to half full load. Thus, starting up with a full head-pond and this inflow, the station can be run at full load for 3 hours, and thereafter at half-load so long as the half-load flow can be maintained from Glenlochar. Tongland reservoir is replenished from loch Ken, which when full, and assuming no inflow, has a storage equivalent to full-load running of the station for about 20 hours. The time-lag between the release of water at Glenlochar and its availability at Tongland dam depends on the head in loch Ken, and varies from about 3½ hours with a full loch to about 8 hours with a low loch.

Power-Stations and Plant.

Investigation showed that the scheme would be economically satisfactory if it were laid out to give a load-factor of about 20 per cent. in a year of normal rainfall. Accordingly plant of a rated capacity of 102,000 kilowatts was installed in the five power-stations, of which particulars are given in Table X. The hydraulic conditions at each station are set out in Figs. 31, Plate 3.

Table X.—Particulars of Plant.

Rated capacity 102,000 kilowatts; voltage of generation 11,000.

	Tongland.	Glenlee.	Earlstoun.	Carsfad.	Kendoon.
Range of net heads: feet	114–95	415–346	68–66	66–64	156–147
Number and rating of generating sets installed: kilowatts.	Three, 11,000	Two, 12,000	Two, 6,000	Two, 6,000	Two, 10,500
Turbines:—					
Output: horse-power	15,500	16,800	8,500	8,500	14,800
Specific speed: revolutions per minute.	84	35	104	104	57
Normal speed: revolutions per minute.	214·3	428·6	214·3	214·3	250
Runaway speed: revolutions per minute.	418	813	450	450	463
Water-consumption per turbine on full load at average head: cusecs.	1,415	435	1,250	1,300	950
Diameter of runner	8 ft. 8 in.	5 ft.	9 ft. 2 in.	9 ft. 2 in.	7 ft. 7 in.

In addition there is one 250-kilowatt auxiliary set at Tongland (water-consumption 35 cusecs) and two 500-kilowatt auxiliary sets at Glenlee (water-consumption 22 cusecs each).

A view of the Tongland turbine-room is shown in *Fig. 32* and the general arrangement of the Earlstoun power-station in Figs. 33, Plate 3.

Adjacent to each station is a step-up transformer substation of the Central Electricity Board, to which the power generated in the station is delivered. Switching for the 132-kilowatt substations is performed only at the three manually-controlled power-stations; for Earlstoun and Carsfad substations it is carried out at Glenlee and Kendoon respectively. Fig. 34, Plate 3, shows the main electrical connexions of the Company's system and its interconnexion with the Central Electricity Board's system.

The design of the plant in the power-stations and the general layout

Fig. 32.

TONGLAND TURBINE-ROOM.

Fig. 37.

EARLSTOUN TURBINE-RUNNER.

Fig. 39.

TONGLAND SPIRAL CASING AND BARREL.

of the installation incorporate a number of unusual features, some of which are dealt with below.

Remote-Controlled Power-Stations.

Three of the power-stations are manually operated. The other two—Earlstoun and Carsfad—are unattended and are normally operated by remote supervisory control from Glenlee control-room; they each contain two 6,000-kilowatt sets and are the first important examples of remote-controlled generating stations in the British Isles. They can be operated under:—(*a*) remote supervisory control from Glenlee power-station, with automatic control of the turbines; (*b*) local control with automatic control of the turbines; (*c*) local control.

With each turbo-alternator in the two stations there is provided an automatic controlling equipment suitable for starting, running up to speed, synchronizing, shutting-down and protecting the turbo-alternator set. When the station is under remote supervisory control from Glenlee the automatic controlling equipment takes charge after the operation of the appropriate control push-button on the supervisory control-board and after the impulses over the supervisory system corresponding to the desired operation have been received. The control-board in Glenlee is equipped with a mimic diagram of the equipment in each station, and with apparatus for giving all necessary indications and performing the requisite operations. Standard-type telephone-relays are employed in the supervisory and automatic equipments, and four pairs of wires are used between the Glenlee control-room and each power-station.

Operation of Glenlochar Barrage.

One of the most important duties of the control-engineer at Tongland is to operate the barrage so that, while flooding of land below the barrage is avoided, water is always available on time and at the highest possible mean head for the load-period at Tongland without any being lost over the spillway of the dam. A system of supervisory control has been installed which enables the engineer to raise or lower any of the six barrage-gates at will. It also gives indication of the position of each gate, of the water-level up-stream of the barrage, and of the flow from the barrage. These indications are obtained from potentiometers mounted on the gate-operating mechanism and on floats respectively. In addition, audible and visible alarms are given in the control-room when the water-level on the up-stream side of the barrage has risen 6 inches above the waste-weir level.

The gates can also be raised by hand or electrically by push-button control on the barrage itself. They are also raised automatically when the water-level in the loch has risen above the weir-level.

Turbines.

The main generating sets are vertical machines. For Tongland, Glenlee and Kendoon, where the specific speeds are moderate or low, the runners are of a modern Francis design. The Earlstoun and Carsfad turbines, on the other hand, have a fairly high specific speed (104 revolutions per minute) and for them a type of runner (Fig. 35, Plate 3) has been adopted which is intermediate between the Francis and the pure propeller type.

Turbine-Runners.

There was ample experience on which to base the design of the blading for the Tongland, Glenlee and Kendoon runners, but on account of the relatively high specific speed of the runners for Earlstoun and Carsfad it was essential to check their design by testing a scale model. When the contract for these sets was placed there were no facilities in Great Britain for carrying out such a test, and the makers decided to instal at their Works an hydraulic testing plant in which brake tests of a complete model of the Earlstoun–Carsfad turbine, comprising distributor, runner and draught-tube, to a scale of approximately 1 : 4, were carried out with exactness. It had originally been intended that the Earlstoun–Carsfad runner should have a skirt like the Tongland runner, but the tests showed that with the same blading a skirtless runner gave a better efficiency and a higher output. Fig. 36, Plate 3, shows the extent to which the performance of the full-size runner differs from the results of the model tests. One of the completed runners is shown in *Fig. 37* (facing p. 383).

Generators.

The alternators at Glenlee run at a comparatively high speed and have rotors of relatively small diameter and great length. The machines therefore are of the usual three-bearing type with a combined thrust- and guide-bearing above the rotor, one guide-bearing below the rotor, and another on the turbine-shaft. All the other machines have alternators of the umbrella type (Fig. 38, Plate 3) with the Michell-type bearing on the shaft below the rotor centre. The rotor has a dropped rim so that its centre of gravity is nearly in the plane of the thrust bearing. These are the first machines of this type built in Great Britain; they have the advantages of requiring

only two bearings, of needing less head-room for dismantling, and of allowing a lighter construction than the more usual type.

Closed Air-Circulation in Alternators and Exciters.

For the first time in normal European hydro-electric practice, closed air-cooling has been provided for the alternators. With the low and medium shaft speeds of water-turbine-driven sets open air-circulation has been the universal rule, but the Galloway alternators have been arranged for closed air-circulation, even to the inclusion of the direct-coupled exciters. A set of fan-blades is mounted on each face of the rotor, and the exciter has its own fan. The air is cooled by water taken from the spiral casing through a reducing valve. The air can be kept above the dew-point when the machine is standing by thermostatically-controlled tubular electric heaters.

The closed air-system was adopted to secure freedom from noise and dirt, and to avoid damage to the windings due to condensation of atmospheric moisture in the damp climate of Galloway on machines which are normally in use only during hours of peak load.

Generator Supporting Barrels.

A coned support of steel plate surrounded by a reinforced-concrete pedestal carries the alternators. This supporting barrel is stiffened by horizontal and vertical ribs to assist in resisting the short-circuit torque of the alternator. It transmits the weight of the stator and the revolving parts, which at Tongland may reach 140 tons (including the hydraulic thrust) to the speed-ring of the spiral casing and through it to the foundations. *Fig. 39* (facing p. 383) shows the barrel for one of the Tongland machines mounted on its spiral casing.

Use of Welding and of Stainless Steel.

Electric welding was employed to an unusual extent in the construction of the plant. The draught-tubes were built up of butt-welded steel plates. The spiral casings for all the stations, except Glenlee, were fabricated of mild-steel plates and welded in four sections to the speed-ring. The sections were fixed together with temporary fastenings before dispatch so that the generator-barrels could be assembled on them. The casings were then dismantled and shipped to site, where they were re-assembled by means of butt and stitch welding. When welding was completed hydraulic pressures varying from 90 to 330 lb. per square inch at the different stations were applied, and any leaks that became visible on tapping with a hammer were caulked or welded. Experience showed that caulking was more effective than welding where the leak was not due

to the splitting of a welded joint, and the repair could readily be tested by applying a blow-lamp to the repaired part with the casing full of water under low pressure. The concrete was placed around the casing after it had been made watertight under the test pressure.

The generator support-barrels were fabricated by welding, as were the lower brackets and the oil-bath containing the Michell-type bearing. In the machines themselves welded parts were largely used instead of castings, for example, the top and bottom turbine-covers with their intermediate compartments, the bearing-housings and the stator-frame.

Some rather unusual welding work provides rustless surfaces on the inner bodies of the cylindrical balanced valves and relief-valves. These inner bodies are steel castings, and the outer cylindrical surface which forms a guide for the sleeve constituting the closing organ of the valve was coated with stainless steel in the shape of a strip 1½ inch wide and ¼ inch thick wound on spirally and electrically welded to the body, which was then turned down to final size. Similarly, strips of stainless steel were welded on the outer edges of radial ribs cast on the inner body to provide guides on which the sleeve-piston slides.

It was desired to use stainless steel for the Earlstoun and Carsfad runners, where cavitation conditions are severe, but the cost of complete runners in that material would have been prohibitive. As, however, the risk of cavitation is to a great extent confined to the backs of the runner-blades near the outlet-edges, protection was provided by the following methods. For the Kendoon runners the critical areas of the back of the blades of the cast-steel runner and adjoining portions of the skirt were ground and coated with stainless steel by electric welding, using stainless-steel electrodes. The welded surfaces were afterwards finished smooth by grinding. For the Earlstoun and Carsfad turbines the cast-steel runners were cast without the trailing-edge portions of the blades, these portions being made as stainless-steel forgings and electrically welded on to the main casting. The blade-surfaces were then finished smooth throughout by grinding.

Balancing of Turbine-Runners and Alternator-Rotors.

A special method was adopted for balancing the runners for Earlstoun, Carsfad and Kendoon. Each runner was mounted with its vertical centre-line on the table of a boring mill and supported on a steel ball above its centre of gravity. When the table was rotated any out-of-balance was shown by the tilting of the runner under centrifugal force. The alternator-rotors were balanced in the same way.

Dimensions of Surge-Towers.

The dimensions of the surge-towers at Tongland, Glenlee and Kendoon were determined by the consideration that when all machines at any of the stations are supplying their full output over one Grid circuit, either to the north or the south, the tripping of a switch will throw off the whole load and the turbines will reject full-load flow. The towers have to be of sufficient capacity to take the resultant upward surge without risk of spilling over the top. There must also be no danger of the downward surge being so great that air is drawn into the pipes when the turbines are put on load again. Further, the diameter of the tower must be such that the surges will be damped out rapidly. Provision was therefore made when designing the surge-towers for 100-per-cent. load-changes for loading as well as for unloading. Usual practice provides surge-tower accommodation for 100-per-cent. full-load reduction but only for 60 per cent. of full load being thrown on; the ampler provision made in this case is desirable in order to provide a margin for the risk of a superimposition of surges resulting from two load-changes in rapid succession.

Screen-Cleaning Gear.

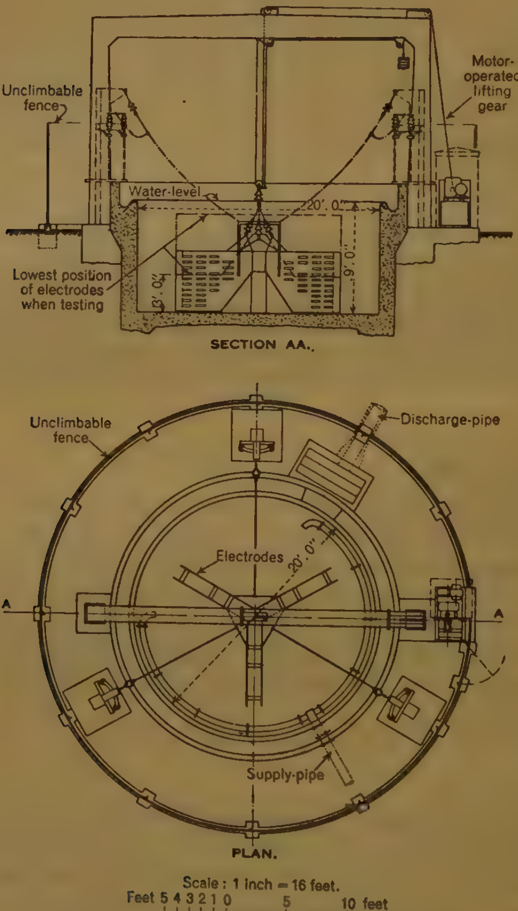
At Tongland considerable quantities of debris come down from the comparatively well-wooded shores of loch Ken and the river Dee, and motor-driven screen-cleaning gear was provided at the intake. Screen-cleaning machines have not been installed at the three upper stations, nor at Clatteringshaws intake, since the country is mostly bare moorland with few trees and it was not expected that much floating debris, other than heather and grass, would collect on those intake-screens.

Test and Loading Ponds.

Water-resistance ponds (*Figs. 40*, p. 388) are provided at Tongland, Glenlee, and Kendoon, so that machines can be tested and governor-adjustments made on steady loads and without interfering with the Grid system. Such adjustments are more important in hydro-electric than in steam stations, since excessive water-hammer and surges caused by incorrect governor-setting may produce serious failures. The output of the machines at Earlstoun and Carsfad can be transmitted respectively to the Glenlee and Kendoon ponds over the Company's 11,000-volt line. Each pond is capable of absorbing continuously 25,000 kilowatts at 11,000 volts between phases. Special test-pond busbars are provided on the main switchgear, and each main alternator circuit-breaker and the Earlstoun and

Carsfad feeder circuit-breakers at Glenlee and Kendoon respectively are of the double-busbar type so that they can be plugged into the main busbar or the test busbar as desired.

Figs. 40.



TYPICAL ARRANGEMENT OF TEST TANK.

TRANSMISSION-LINES.

When the constructional works were being started it was arranged that any electric power required by the contractors should be supplied by the Central Electricity Board, who expedited the construction of the Grid line between Dumfries and Kilmarnock for this purpose.

The Company's overhead-line programme was planned with the same object. The first line to be erected ran along the route of the Glenlee tunnel (with an extension to the site of the Clatteringshaws dam) from a temporary substation put up at Glenlee by the Central Electricity Board and containing a 3,000-kilovolt-ampere transformer stepping down to 11,000 volts and fed through a 132,000-volt fuse. The line was looped into four construction substations on the tunnel. The Central Electricity Board also installed a temporary substation at Tongland, from which supplies were given to the contractors on the power-station site and through the Tongland–Glenlochar line to those at Tongland dam. At a later date the Central Electricity Board's transformer and high-tension fuse unit were transferred to a temporary substation at Drumjohn, where they were used to give a supply to the work on the Doon and Deugh tunnels and to the contractors' camp there.

The five power-stations are linked together by the Grid line, but it was necessary to have another connexion between the stations, since the Kirkcudbright County Council is entitled to supplies from two of the power-stations and from three points on the Company's transmission-lines. Accordingly, an 11,000-volt line (Fig. 34, Plate 3) was run from Tongland to Kendoon, looping into the intermediate stations. Between Tongland and Glenlochar barrage the line is double-circuit with two open telephone circuits and two supervisory-control wires. Between Glenlochar and Glenlee it is a single circuit without telephone or supervisory wires. From Glenlee to Kendoon the line is a single circuit with a catenary carrying a telephone and pilot cable for supervisory control of the Earlstoun and Carsfad power-stations and for indications of the meters and instruments of the Company and the Central Electricity Board.

Table XI (p. 390) gives particulars of the Company's transmission-system.

Safety-Devices and Protective Measures.

Electrical or mechanical devices are provided to protect the different items of plant from injury due to accident or incorrect operation. In addition, various measures were taken to reduce the damage to the plant which may result if any part of the installation fails to function correctly.

Generating Plant.

The turbo-alternators are protected against overspeed, underspeed, loss of governor or lubrication oil-pressure, and excessive temperatures. The unattended power-stations are protected under

TABLE XI.—OVERHEAD TRANSMISSION-LINES.

	Glenlee–Clatteringshaws.	Tongland–Glenlochar.	Glenlochar–Glenlee.	Glenlee–Kendoon	Kendoon–Blackwater Burn dam.
Voltage	11,000	11,000	11,000	11,000	11,000
Length of route: miles	6·25	9·35	13·7	4·73	0·72
Number of circuits	One.	Two.	One.	One.	One.
Area of conductor (copper): square inch	0·058	0·1	0·15	0·15 (2·34 miles) 0·05 (2·39 ,,)	0·05
Type of supporting pole	Single, wood.	"A," wood.	Single, wood.	"A," wood.	Single, wood.
Span (maximum): feet	350	350	310	360	350
Auxiliary wires	Two open wires Cd-Cu. No. 10 S.W.G.	Four open wires Cd-Cu. No. 10 S.W.G.	None.	Catenary cable; 6-core 14-pair (G-E), 3-core 7-pair (E-C), 6-core 3-pair (C-K).	Two open wires.
Cost: £	3,555	13,438	9,136	9,956	
Cost per mile: £	568	1,440	666		

NOTES:

G–E signifies Glenlee to Earlstoun, E–C Earlstoun to Carsfad, C–K Carsfad to Kendoon.

The catenary wire supporting the pilot and telephone cable is 7/8 S.W.G. steel. It acts also as an earth conductor.

Pilot cores are 7/·029 and telephone cores 20 lb. per mile.

supervisory control by devices which, if abnormal conditions arise, prevent the sets from starting-up or shut them down if they are running.

Precautions against Flooding.

A disaster occurred in a hydro-electric installation on the Continent in January, 1934, when a manhole-cover on a pipe-line burst and released a jet of water which flooded the power-station, drowning nine men and wrecking the transmission and communication circuits; as a result there was delay in closing the intake-gates and a city farther downstream was in imminent danger of being flooded.[1] This emphasizes the importance of being able to close promptly in emergency the intake-gates of the tunnels and aqueducts. All intake-gates in Galloway (except that at Tongland dam, which has a duplicate electric supply) were designed to close by gravity against full flow with maximum head in the reservoirs. They can be tripped either—

(*a*) automatically by paddles in the aqueducts, which operate when the flow becomes excessive,
(*b*) locally by hand,
(*c*) electrically from the power-stations.

At Tongland the operating circuits for electric tripping were run in duplicate by separate routes and were arranged to take their supply from two independent sources.

Precautions against Fire.

A water-power station might not be expected to take fire, but since there is a considerable amount of oil in the switchgear and the turbine governor-system some fire-fighting equipment is desirable. This takes the form of hydrants inside and outside the buildings, and hand and portable extinguishers. Oxygen-breathing apparatus is provided to enable men to work in the clouds of smoke that would be produced by burning oil. The switch-houses are divided into compartments with self-closing doors having small openings through which the nozzle of a chemical fire-extinguisher can be inserted. Teak sills are fitted to the doorways to prevent burning oil passing from one compartment to another. Transformers and reactors are set on raised plinths filled with granite chips and provided with drainage to soak-aways outside the building. Buchholz relays are fitted to the indoor transformers and reactors.

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[1] "Zur Rohrbruch-Katastrophe am Schwartz-See." *Schweizerische Bauzeitung*, vol. 103, p. 59 (3 February, 1934).

Precautions against Frost.

The winters in Galloway are not severe, and consequently the precautions against frost are not elaborate. Pipes for electric heating elements are fixed in the gate-grooves at Glenlochar barrage and Tongland flood-gates, and provision is made for a supply of low-pressure air to prevent ice forming on the grid in front of the Clatteringshaws intake if the weather should be frosty when the level of the loch happens to be low. Heating plugs are provided in the valve-houses and in the cylindrical balanced-valve pits in Glenlee power-station. The probability of any appreciable amount of frazil ice forming was considered so remote that no provision was made for heating the intake-grids.

Precautions against Lightning.

The Galloway district is subject to severe thunderstorms; the Grid line, being insulated for high voltage, is not often put out of action, but the 11,000-volt lines throughout the county are shut down comparatively often. Methods adopted in the Galloway scheme for reducing the risk of trouble from lightning include the provision of lengths of underground cable between the switchgear and the overhead-line terminations, the grading of insulators on the first few spans of the lines, and the installation of surge-absorbers, expulsion-gaps and lightning-arresters.

Operating Experiences and Results.

The power-stations are "Selected Stations", and all the energy produced (except a small quantity for station-auxiliaries and compensation-supplies) is taken by the Central Electricity Board for distribution over the Grid. Thus the stations are operated to the directions of the Board, and the Engineers of the Company and the Board work together to ensure that water is available when required and that load can be taken when there is a prospect of losing units by spilling at the dams; the successful results obtained since operation started are largely due to close collaboration on these matters.

The scheme was designed to give a maximum output of 102,000 kilowatts, but during the first 6 months of operation the maximum load was frequently 106,000 kilowatts; even if the daily-storage reservoirs are not filled to capacity when there is an unexpected call for maximum output, at least 100,000 kilowatts can usually be given within a short time. The output during the first

12 months of full operation was 242 million units, which is considerably above the estimate on which the scheme was based.[1]

In addition, it has been demonstrated that the scheme possesses certain other valuable features which, although not readily expressible in money, are in fact very real; these include:—

(i) Quick starting-up both normally and in emergency; at Glenlee alone 24,000 kilowatts of power are always available within a few minutes, and in the whole scheme about 100,000 kilowatts at comparatively short notice.

(ii) Quick shutting-down when the industrial load is suddenly reduced, without loss of water and without the complications inherent in steam plant.

(iii) Maintenance of the voltage on the long transmission-line between Glasgow and Carlisle.

(iv) Regulation of the frequency of the Grid in Scotland and N.W. England.

(v) Ability to supply cheap units during the summer when efficient steam plant is being overhauled.

Table XII gives the actual times required for putting the stations on load.

TABLE XII.

TIME REQUIRED FOR PUTTING STATIONS ON LOAD.

Time: minutes.	Load: kilowatts.					Total.
	Tongland.	Glenlee.	Earlstoun.	Carsfad.	Kendoon.	
5	11,000	12,000	—	—	11,000	34,000
10	22,000	24,000	—	—	22,000	68,000
15	33,000	24,000	—	—	22,000	79,000
20	33,000	24,000	6,000	6,000	22,000	91,000
25	33,000	24,000	12,000	12,000	22,000	103,000

All load can be dropped in 5 to 10 minutes.

In any pioneer installation such as this, however carefully it may have been thought out, there is always the possibility that some unforeseen difficulties may arise or that some well-tried engineering

[1] The levels of lochs Doon and Clatteringshaws were somewhat lower at the end of the 12 months than at the beginning. The difference in levels represented about 9 million units.

features may react unexpectedly to new conditions. Among the questions which only experience could answer were the following :—

(1) What would be the effect of introducing a group of water-power stations into the middle of a long transmission-line connected at each end to groups of steam-power stations ?

(2) Would any complications arise owing to the different characteristics of the governors of the water-turbines and the steam-turbines ?

(3) Would there be trouble due to load-swinging ?

(4) Would oscillation of water-pressure in the pipe-lines or tunnels make synchronizing difficult ?

(5) Would there be any difficulty in dividing the load as desired between Scotland and North-West England ?

(6) What would happen when all the stations were giving full output if the line opened at one or both ends ?

Experience has shown that little need be feared on any of these grounds ; in fact no unexpected complications have arisen and operation has proved simpler than was anticipated.

Apart from unexpected calls for power by the Central Electricity Board to meet special conditions on their system, the working of the scheme falls naturally into the two categories of normal day-to-day operation and operation under flood-conditions.

Normal Day-to-Day Operation.

Generally speaking, the scheme is so flexible that it is neither desirable nor necessary in normal conditions to operate it to a rigid load-programme. In fact such a proceeding, although it would simplify operation from the Company's point of view, is undesirable on the wider consideration of the efficient operation of the National Grid. The day-to-day output from the scheme is generally arranged a short time only before the actual load is required. At Tongland, however, somewhat longer notice is needed on account of the time required for the flow of water from loch Ken. In general, the regulation of flow must have regard to the following :—

(i) The anticipated load-requirements of the Grid.

(ii) The amount of water available in the main reservoirs (lochs Doon and Clatteringshaws), and particularly in the daily reservoirs and the rivers.

(iii) The prospect of heavy rain—this is of special importance towards the end of a week because the requirements of the Grid are then a minimum.

(iv) The necessity or otherwise of being able to give additional emergency load to the Grid.

(v) The desirability of avoiding loss of units by spilling at the dams.

The Central Electricity Board's requirements are generally stated in the form of so many kilowatt-hours of power for two or three periods during the day, and the programme for the regulation of the water is arranged accordingly. Normally, the flow of water is regulated so that the head-ponds are full at about 7.30 a.m., in time for the morning peak load. It might be thought that this would be difficult to arrange, but in point of fact it is comparatively easy with a little experience. Occasionally, however, it is found that, owing to a local thunderstorm, one or more of the head-ponds fill up more rapidly than was calculated and there is a danger of water being spilled. Fortunately the flexibility of the Grid system usually permits the Central Electricity Board to take at short notice the load necessary to avoid such spilling until the normal industrial load has to be dealt with.

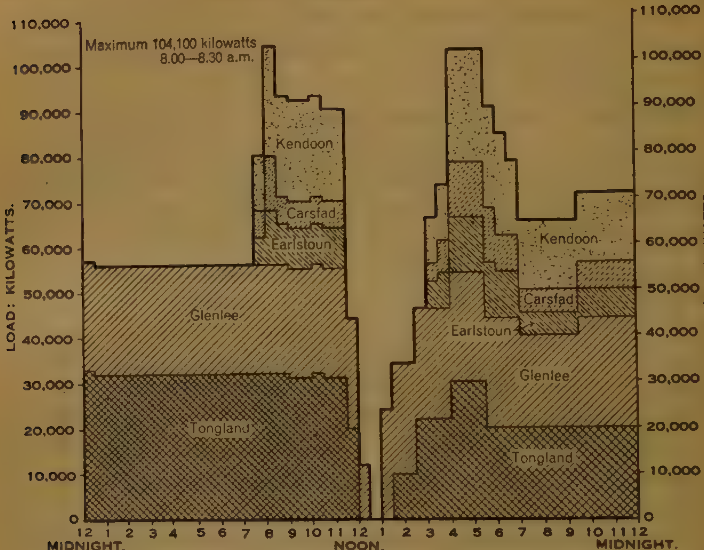
If the load-programme does not call for the full output of the scheme, the four stations, Kendoon, Carsfad, Earlstoun and Tongland, which take the run of the rivers are operated at or near full capacity as long as their reservoir-storage and the unregulated water-flow permits, Glenlee power-station being held in reserve for "topping up" and emergency operation.

The flexibility of the scheme from the Central Electricity Board's point of view is indicated by *Figs. 41* and *42* (p. 396) which show the loads supplied to the Grid on the 15th January and the 13th October, 1937. On the former date there was a flood, and the stations were run to give all the output the Grid could absorb so as to avoid loss of units by spilling. On the latter date no water was going to waste and the Board took only the power required to help it to deal with the peaks. The load-factors on these two days (relative to the total plant installed) were 65·5 and 6·2 per cent. respectively.

Operation under Flood-Conditions.

It has been realized ever since the hydraulic conditions were first investigated that it would be impossible during a heavy flood to avoid entirely the spilling of some water at one or other of the river stations. Accordingly, as soon as it is known that a flood is approaching, steps are taken to reduce spilling to a minimum. Arrangements are made with the Central Electricity Board to give the load necessary to draw down the head-ponds as much as possible so that the prearranged load-programme may be dealt with by the flood-water.

Fig. 41.

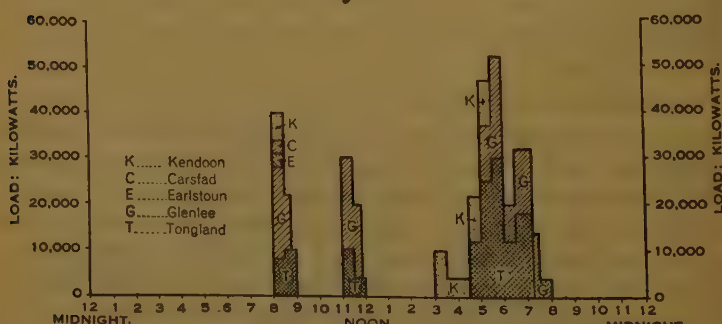


Load-Curve of System: 15th January, 1937.

As soon as possible the valve at Drumjohn is closed and the maximum amount of water from the upper part of the Deugh is stored in loch Doon. Also, as soon as load-conditions permit, Glenlee power-station is shut down or its load reduced to store as much as possible in Clatteringshaws loch. These measures assist in reducing the water to be dealt with in the river.

Generally the first warning of a flood is given by the attendant at Drumjohn to the Kendoon power-station staff, who, in addition to passing the information on to Glenlee and thence to Tongland, have

Fig. 42.



Load-Curve of System: 13th October, 1937.

to decide how soon the inflow from loch Doon or the river Deugh through the Drumjohn valve may be closed down having regard to the ordinary load-commitments. It takes about a day for a heavy flood to travel down the rivers from the upper part of the catchment-area to Tongland, but owing to the unregulated water which comes into the river from the east and west and to the necessity of keeping reservoir-storage to deal with the ordinary load-programme, the notice even at Tongland may be a matter of a few hours only. At Tongland, it is usual to prepare for an oncoming flood by drawing down loch Ken as quickly as possible after notice is received. In this way an artificial flood is produced in the Dee about a day in advance of what would normally be the natural flood, and the latter, if of short duration, can be held in loch Ken.

Between November 1936 and November 1937, the number of units lost by water spilling over the dams was 10,171,660.

One of the advantages expected from the scheme was that the tops would be taken off the natural floods, so that damage to land adjacent to the river would be reduced. How fully this has been realized can be seen from *Figs. 43* (p. 398), which show the actual rise of the river at Glenlochar during two floods in the winter of 1936–37 and the heights to which the river would have risen if the flow had not been regulated.

Floods and Weather-Conditions.

The Galloway rivers and streams rise and fall very rapidly, and this makes it difficult to avoid spilling water, particularly if, as often happens, storms arise in the night or at the week-ends, when the Grid load is comparatively small. Watching the weather-conditions is therefore an important part of the operation of the scheme, and the recording barometer is valuable as a guide in this respect.

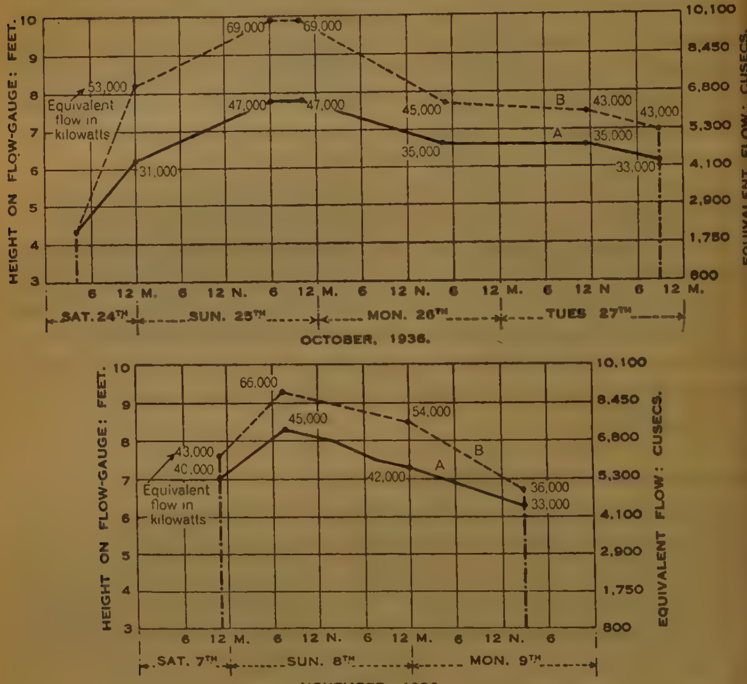
The valve-attendants at Drumjohn, Blackwater, Carsfad and Earlstoun, Clatteringshaws and Glenlochar report the weather-conditions daily to Tongland, and so do the staffs at Kendoon and Glenlee. The level of Kendoon reservoir, which is affected not only by the inflow from Drumjohn but also by any water spilt at the Deugh intake and by the inflow from the Water of Ken and the Black-water Burn, is indicated in Kendoon control-room. The levels of Carsfad and Earlstoun reservoirs, which receive the unregulated flows of the Polmaddy and the Polharrow Burns respectively, are indicated in Glenlee control-room.

Water-Regulation and Availability for Load.

To be able to work to a pre-arranged load-programme, it is necessary to regulate the water so as to have the correct amount available

in the head-ponds at the right times. At Glenlee this presents no difficulty, as this station in effect has water always available; although this station is designed as an integral part of a peak-load scheme and is generally so operated, it can be operated as a base-load station when occasion demands, running at full load for several

Figs. 43.



Notes:—
Curves "A" show height of river flow at bridge with storage in Loch Doon and Clatteringshaws Loch
Curves "B" show estimated height of flow with no storage

RECORDED RIVER-FLOWS AT GLENLOCHAR BRIDGE DURING FLOOD PERIODS (NATURAL FLOW WITH BARRAGE-GATES LIFTED CLEAR OF WATER).

weeks. Even if there were no inflow to Clatteringshaws loch, the station could run at full load for 16 or 17 days if the loch were full at the start.

A small complication in the regulation of the water is introduced by the by-pass water which has to be discharged continuously down the fish-ladders at the dams at Carsfad, Earlstoun, and Tongland, and at Glenlochar barrage. At the three dams this is approximately

12 million gallons per day, but at the barrage it is substantially greater and varies with the level of loch Ken; with a normal loch and with the barrage-gates closed this flow is sufficient to replenish Tongland reservoir in about 36 hours, that is, over the week-end if the station is shut down. Another similar complication arises from the drainage and run-off from the land adjoining the rivers. In a dry spell quite a heavy shower has no effect, whilst a much smaller shower after wet weather causes appreciable inflow.

It will be appreciated that normally Kendoon, Carsfad and Earlstoun are operated simultaneously. If one machine only is run at Kendoon, the other two stations also run with only one machine each. The reservoir-levels at Carsfad and Earlstoun can be adjusted by varying the load between the stations; with average river-flow 24,000 kilowatts at Kendoon, 10,000 kilowatts at Carsfad, and 12,000 kilowatts at Earlstoun will maintain the levels steady.

Storage of Water.

The levels maintained in the storage-reservoirs require consideration and attention. They are brought as high as possible at the beginning of the summer and are drawn down during the late summer in anticipation of the autumn and winter rains; at the same time, as the occurrence and length of any drought cannot be foreseen, it is undesirable to draw down too far if emergency storage has to be maintained. Both in winter and summer, on the other hand, the reservoirs must not be allowed to become too full, otherwise a heavy storm may cause spilling and a heavy loss of units.

OPERATING TROUBLES.

An account has been given earlier in this Paper of the measures taken to minimize the effect of the troubles that were to be expected. Experience has already shown that many of these measures were necessary.

Lightning.

The main troubles which have occurred were directly and indirectly due to lightning. The main difficulty, however, is, not the number of times power-lines have been struck, but the fact that the Post Office telephone-system is frequently disorganized directly or indirectly by falling trees. As that system is depended on for communication not only with the Central Electricity Board's control-room in Glasgow, but also to a large extent on the Company's system, transmission of messages during storm periods has been rendered difficult and slow.

It is hoped that most of this trouble will disappear with the completion of the undergrounding of the Post Office main lines, which is now proceeding.

The actual cases of damage due to lightning over a period of about 5 years, including about 2 years of operation, have been :—

(1) one pole burnt in the Clatteringshaws line ;
(2) about a dozen breakages of overhead-line insulators ;
(3) four cable-box breakdowns at terminal poles ;
(4) five instances of lightning damaging switches, and two cases of cables breaking down in power-stations.

Trouble from lightning is increased by the fact that it is practically impossible to obtain low earth-resistances, owing to the rocky nature of the ground. This complicates the operation of the protective gear.

Snow and Frost.

As in the remainder of the country, trouble was experienced in Galloway during the very severe blizzard of February, 1933. On the Clatteringshaws 11,000-volt overhead line one spur about a mile long was brought within about 4 feet of the ground with a snow-sleet load of about 4 inches diameter on the conductors. Contrary to expectations and calculations, most of this line apparently recovered its original sag, and the damage might have been confined to a few broken insulators and some bent ironwork had not some of the copper become annealed owing to the lines swinging together, and the protective gear having been rendered inoperative to maintain the supply. When the snow disappeared it was found that one section which had been kept alive on the ground had actually burnt the ground and rock for several yards. No poles were broken, presumably owing to the boggy nature of the ground, which allowed them to give a little.

Screens were fitted in the Glenlee tail-race where it discharges into the river Ken to keep out salmon. In times of severe frost, if the station is shut down throughout the night the shallow pools of water left in the tail-race freeze solid, and when the station starts up in the morning, the ice, sometimes 2 inches thick, is swept down against the screens and piles up in a solid mass which may be 4 to 6 feet thick. This causes flooding from the tail-race till the ice is cleared away. A spillway provided on the upstream side of the screens relieves the pressure caused by the blocking of the screens.

Ice which forms at Tongland dam is prevented from causing trouble at the intake-screens by letting extra water down from loch Ken so as to discharge the ice over the spillway, and occasionally by running up a machine when the station is normally off load. This also pre-

vents the possibility of trouble in the valve-house and in the surge-tower.

Trash and Debris on Screens.

Immediately after the completion of the scheme there was an appreciable amount of contractors' scrap material floating down the river, but otherwise very little trouble has been experienced with debris. In the autumn, particularly after the first heavy flood, a large amount of leaves and other debris comes down, and men have to be specially detailed for screen-cleaning for a couple of days or so.

Plant and Automatic Equipment.

The water in the storage-reservoirs and rivers is normally free from sand or silt, and a recent inspection of the turbine-runners showed that they were in excellent condition, wear due to erosion and cavitation being very small. Some slight wear has taken place on the Tongland and Glenlee runners, which were operated at first for long periods at about one-quarter and one-half of full load. The machines are now run on as high loads as the Central Electricity Board can provide, and never for any length of time below about one-third load.

In the general operation of the plant, there was a surprising lack of even small "teething" troubles, in spite of the fact that the application of a considerable amount of the equipment was novel. This is specially noteworthy because there was no preliminary running by the Company's staff before the plant was put on commercial load after the Contractors had handed it over. With the switching-in of the second set at Carsfad at about 2.30 p.m. on the 27th October, 1936, the scheme was completed. Within 3 hours of this the Central Electricity Board asked for and obtained the full available output of all the stations, which, with the existing water-conditions, was 99,000 kilowatts.

In particular, the automatic stations were unexpectedly free from troubles from the start. There was naturally some anxiety about the reliability of the two remote-controlled automatic stations which were put into commission at the beginning of a winter in which an unusual number of storms occurred. These stations had to commence operation before the remote-control gear was completed, and so were at first manually operated. When the remote-control gear was available they were operated by it, but as the output of these stations was essential to assist the Central Electricity Board to deal with the load-demand, a skeleton staff was kept standing by in the stations until the winter peak-load was over. Experience soon

showed that this precaution was unnecessary, and the routine now established is that these stations are started and stopped entirely by remote control, and run without any attendance at all except for the routine daily visits of the maintenance staff or when handymen are doing cleaning work or filtering oil. Table XIII gives a list of the troubles experienced during the first 10 months of operation of these stations.

TABLE XIII.—LIST OF FAULTS AT CARSFAD AND EARLSTOUN WHICH HAVE CAUSED SETS TO BE OUT OF COMMISSION DURING THE 10 MONTHS IN WHICH THE STATIONS HAVE BEEN ON COMMERCIAL LOAD.

Nature of fault.	Number of faults.	Total.	Duration of fault, maximum and minimum.*
Mechanical Faults :			
Air-brake relay sticking	3		10–30 minutes.
Solenoid oil-valves leaking	3		15–90 ,,
Choked oil-pipe	1	10	60 ,,
Faulty pipe-valve	1		150 ,,
Intake-gate sticking	1		150 ,,
Intake-gate limit-switches	1		40 ,,
Electrical Faults :			
Speeder motor armature binding	1		180 ,,
Potential-transformer fuses blown	2		10 ,,
Potential-transformer winding burnt out	1	19	Several days
Voltage-regulator	6		20–120 minutes
Contactors not making	2		33–60 ,,
Mercury-pressure relays sticking	7		15–120 ,,
Supervisory Faults :			
Relays, etc. operating incorrectly	6	6	10–60 minutes.
Operating Faults :			
Merz-Price relay tripping while synchronizing	4		15–85 minutes.
Lock-out due to apparatus being operated in wrong sequence	2	8	20 ,,
Lock-out due to fuses being withdrawn	2		30–80 ,,

* These periods relate to the time necessary to rectify the fault or replace the apparatus, and not necessarily to the time the stations were out of commission.

It is perhaps somewhat early to say much about the reliability of these stations, but, as Table XIII shows, it is possible to speak highly of the way they have operated so far.

Weather and Incidence of Storms.

Hitherto the weather-conditions have proved more severe than was anticipated from local information, wind, flood, snow and frost

having all been of greater severity. It has not, however, been necessary to make use of the provision made for dealing with frost on the bulk of the plant. It has, however, been found desirable to provide local heating on various valves and electrically-operated equipment in exposed places; this has generally been done by providing small metal-clad electric heating units. Although, generally speaking, frost and ice have not caused any trouble on the main equipment, some small valves and pipes have been damaged; these have now been protected by means of a pipe-insulating compound. The most noticeable effect of frost is to reduce appreciably the amount of run-off and thus lower the levels of the rivers. In the winter of 1935/36 there was a period of several weeks during which this effect was particularly evident.

It is noticeable that during each of the 3 years of operation, there have been storms at certain definite periods—one storm at the end of June or the beginning of July and another storm at the end of October or the beginning of November. During the winter of 1936–37, although Galloway did not suffer from floods as severely as some other parts of the British Isles, the rapidity with which the floods rose was very marked. On the 13th/14th December, 1936, shortly after the start of the operation of the full scheme, there was a particularly heavy flood during which, it is believed, the upper reaches of the rivers had their greatest recorded rise for about 100 years. The flood was due to heavy rain and a sudden thaw which melted the snow that was lying on most of the hills in the watershed. The maximum flow past Earlstoun power-station, about 21,800 cusecs, was nearly nine times the flow through the station when working on full load. The level of loch Ken rose 4 feet 8 inches in 7 hours, and at the peak of the flood the flow at Glenlochar barrage with the gates fully opened was about 10,300 cusecs. The hydraulic works stood up very satisfactorily to these very severe conditions, and the only trouble experienced was a certain amount of erosion due to the floods overtopping the retaining walls of the spillway-channels.

Staff.

In addition to the staff required for the operation and maintenance of the plant and works, some men will be needed to look after the numerous pieces of ground which the Company acquired in connexion with the works. Including these, the permanent staff will probably be between seventy-five and eighty. At present the permanent staff in Galloway numbers sixty-nine, including for management and administration six; for operation forty-eight; and for maintenance fifteen.

Efficiency-Tests.

The efficiency of the alternators was determined in the works in the usual way, the figures given in Table XIV for one of the Kendoon machines being typical.

Table XIV.—Efficiency-Tests of Kendoon Alternator.

Percentage of full load.	100.	80	60	40
Iron loss : kilowatts . . .	102	102	102	102
I^2R+strays : kilowatts . .	117	75	38	15
Excitation : kilowatts . .	60	47	36·2	28
Friction and windage : kilowatts	97	97	97	97
Exciter loss : kilowatts . .	11·5	9·2	7	5·5
Total loss : kilowatts . .	387·5	330·2	280·2	247·5
Output : kilowatts . . .	10,500	8,400	6,300	4,200
Input : kilowatts . . .	10,887·5	8,730·2	6,580·2	4,447·5
Efficiency : per cent. . .	96·4	96·2	95·7	94·4

After the sets had been erected and put into service the combined efficiencies of the turbines and alternators were determined with as great accuracy as was practicable on site. For determining the water-flow, pitot-tube measurements were used at Glenlee and the "salt-velocity" method at the other stations. When all corrections had been made the combined overall efficiencies of the sets at full load were found to be as given in Table XV and Fig. 44, Plate 3.

Table XV.—Combined Overall Efficiency of Turbo-Alternators at Full Load.

Tongland	88·0 per cent.
Glenlee	85·3 ,,
Earlstoun	90·5 ,,
Carsfad	87·0 ,,
Kendoon	88·0 ,,

Costs.

The construction of a steam power-station follows well-defined principles, and it was possible to say, at the time the orders for the Galloway plant were placed, that a modern base-load steam-station could be constructed for a price of £14 to £17 per kilowatt installed. Such a generalization is not possible in the case of hydro-electric stations, since the hydraulic works absorb the bulk of the expenditure (in Galloway nearly 65 per cent.) and the cost of these depends upon the configuration of the land and the local conditions. Thus, in Galloway it was necessary to distribute the plant in five separate

power-stations with their ancillary hydraulic works, instead of concentrating all the plant in one station. The cost per kilowatt of the plant itself depends largely on the speed at which the sets run, and therefore varies within wide limits according to the head of water available. Table XVI gives the cost per kilowatt installed in each of the five Galloway stations. The figures include the turbo-alternators with their accessories, namely draught-tubes, governors, compressors, oiling and cooling systems, and also the main stop-valves or gates and the station cranes; they do not include buildings, switchgear and wiring, penstocks and surge-towers.

The cost of the whole Development as now in service, including hydraulic works, plant, 11,000-volt transmission-lines, land and all expenses, works out at £28·3 per kilowatt of power available.

TABLE XVI.—COST AND PARTICULARS OF GENERATING PLANT, MAIN STOP-VALVES AND CRANES.

	Tongland.	Glenlee.	Earlstoun and Carsfad.	Kendoon.
Average net head: feet . .	106	380	67/65	150
Speed of set: revolutions per minute.	214·3	428·6	214·3	250
Rated output per main set: kilowatts.	11,000	12,000	6,000	10,500
Installed power: kilowatts .	33,250	25,000	24,000	21,000
Cost per kilowatt installed: £	4·1	2·8	5·5	4·0

Cost of Operation.—The full scheme has not been working sufficiently long to provide figures for the cost of operation which would be applicable over an extended period. The results up to the present, however, indicate that the cost will be lower than the estimates on which the scheme was based.

GENERAL.

The scheme was completed in time to fulfil the promise of providing peak-load in the autumn of 1936, a result which should be a matter of satisfaction to all concerned in view of the diverse and scattered nature of the works, the number of different contractors involved, the necessity in many cases of having several contractors at work simultaneously on a restricted site, and the fact that the construction extended over three winters when outside work was at the mercy of the weather; all these conditions necessitated close collaboration between the consulting engineers in directing the work and cordial co-operation among the contractors in carrying it out.

The main Contractors for the mechanical and electrical plant dealt with in this Paper were :—

The English Electric Co., Ltd.	Turbo-alternator sets. Switchgear and auxiliary plant (except Kendoon high-tension switchgear).
A. Reyrolle and Co., Ltd.	Kendoon high-tension switchgear.
Glenfield and Kennedy, Ltd.	Stop-valves and anti-vacuum valves at Tongland.
Clyde Crane and Engineering Co., Ltd.	Overhead electric travelling cranes for Tongland and Glenlee.
Sir William Arrol and Co., Ltd.	Overhead electric travelling cranes for Earlstoun, Carsfad, and Kendoon.
W. T. Glover and Co., Ltd.	Overhead lines.

The Consulting Engineers were Sir Alexander Gibb and Partners for the hydraulic works and Messrs. Merz and McLellan for the mechanical and electrical plant. The Resident Engineers for Messrs. Merz and McLellan were Mr. John Warnock, M.C., Assoc. M. Inst. C.E., on the first part of the scheme (Tongland and Glenlee) and Mr. William S. Gedye on the second part.

Both the Authors, as Engineers on the staff of Messrs. Merz and McLellan, were associated with the work from the inception of the Scheme until the first stations were put to work in the autumn of 1934, when Mr. Williams was appointed Manager of the Company.

The Authors desire to express their thanks to the Galloway Water Power Company and the Central Electricity Board for permission to publish the information given in the Paper.

The Paper is accompanied by fourteen sheets of tracings and three photographs, from which Plate 3, the Figures in the text, and the half-tone page plate have been prepared.

Paper No. 5165.

"The Galloway Hydro-Electric Development, with Special Reference to its Interconnexion with the Grid."†

By Reginald William Mountain, B.Sc. (Eng.), M. Inst. C.E.

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Introduction.

The Galloway district forms part of the area contained in the South Scotland Electricity Scheme of the Central Electricity Board, and the development of water-power in Galloway forms an essential part of that Scheme. The development of the water-power made possible the general supply of electricity to the farming districts of Kirkcudbrightshire and Wigtownshire.

The South Scotland Electricity Scheme covers an area of about 4,308 square miles, and the population in 1921 was under 256,000, the average density being only 59 persons per square mile; most of the area is moorland and forest land. It was decided when examining the South Scotland Electricity Scheme in relation to the development of water-power in Galloway that, after allowing for a general supply of electricity for local requirements, the bulk of the output from the Galloway scheme should be exported, half to Central Scotland and half to North-West England.

The industrial nature of Central Scotland and of North-West England as compared with South Scotland is shown by the following figures. The Central Scotland Scheme has an area of 4,980 square miles, and the population in 1921 was over 3,700,000, the average

† Correspondence on this Paper can be accepted until the 15th July, 1938. —Sec. Inst. C.E.

density being over 740 persons per square mile; the North-West England Scheme has an area of 9,082 square miles, and the population in 1921 was over 6,980,000, the average density being about 770 persons per square mile.

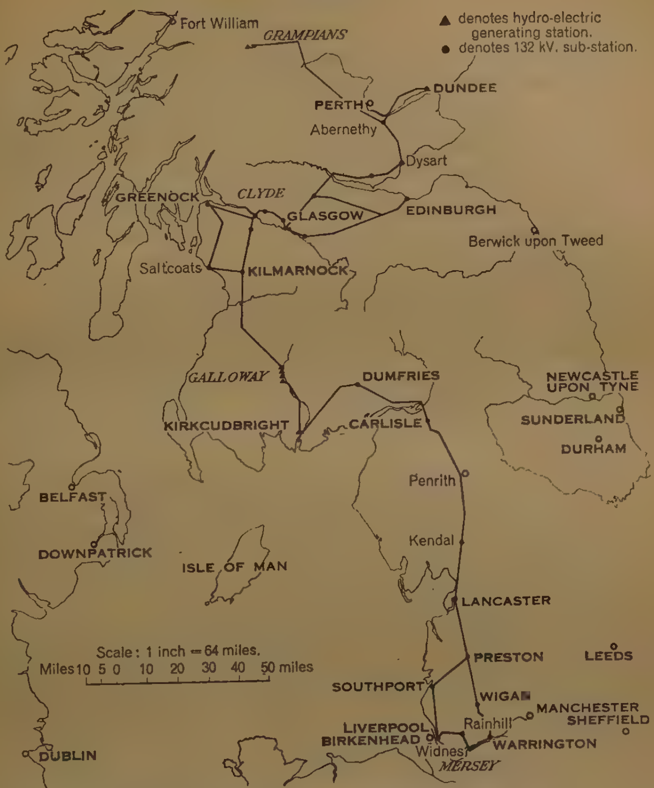
The amount of water available for utilization in Galloway was estimated to be sufficient to provide about 180 million electrical units per annum, and an examination of the cost of the necessary storage-works showed that the design of the scheme would be most economical if it were assumed that the majority of the electricity would be used during the winter months. The scheme was designed for an installed plant-capacity of 102,000 kilowatts, and the corresponding annual load-factor was therefore of the order of 20 per cent. It may be mentioned that the present electrical requirements of Central Scotland and of North-West England together amount to about 4,700 million electrical units per annum, with a maximum load of the order of 1,580,000 kilowatts.

The capital expenditure by the Central Electricity Board on the equipment used for the interconnexion of the Galloway scheme with the 132-kilovolt transmission-systems of Central Scotland and of North-West England was estimated to be about £346,000, out of a total capital expenditure on the South Scotland Scheme of £869,000.

The essential problem was to transmit a load approaching 50,000 kilowatts from Galloway to the industrial area on the Clyde, and at the same time to transmit a further 50,000 kilowatts to the industrial area on the Mersey. It was decided that so far as possible the equipment forming this interconnexion should be of the standard design which had been developed for the 132-kilovolt transmission-system of the Central Electricity Board, and should consist, if possible, of a single-circuit line between Kilmarnock, the most southerly of the substations in Central Scotland, and Carlisle, the most northerly of the substations in North-West England, that transmission-line having been envisaged before the Galloway scheme was adopted. *Fig. 45* shows the relation between the Galloway scheme, the Clyde, and the Mersey, and shows the route finally chosen for the interconnecting transmission-line.

The equipment consists of a single-circuit 132-kilovolt line having a total length of 121 miles, and step-up transforming substations placed adjacent to the hydro-electric generating stations. The 132-kilovolt switchgear controlling these transforming stations has been concentrated at sites adjacent to three of the five hydro-electric generating stations, as this gave the most economical arrangement of plant. *Fig. 46* (p. 410) shows the arrangement of this equipment.

Fig. 45.



132-Kilovolt Transmission-Lines Associated with the Galloway Scheme.

Method of Obtaining Load-Transfer from Galloway to Central Scotland and to North-West England.

The 132-kilovolt transmission-system of the Central Electricity Board was designed on the principle of interconnexion between existing generating stations rather than on the principle of long-distance transmission. It was estimated that only about 20 per cent. of the electricity generated in the country by authorized undertakings would be carried over the transmission-lines, and the remaining 80 per cent. of the output would be sent directly from the generating stations to local distribution-systems. It has been found that many of the transmission-lines are more heavily loaded now than in the earlier years of operation, partly owing to the steadily-increasing load on the whole system and partly because extensions

Fig. 46.

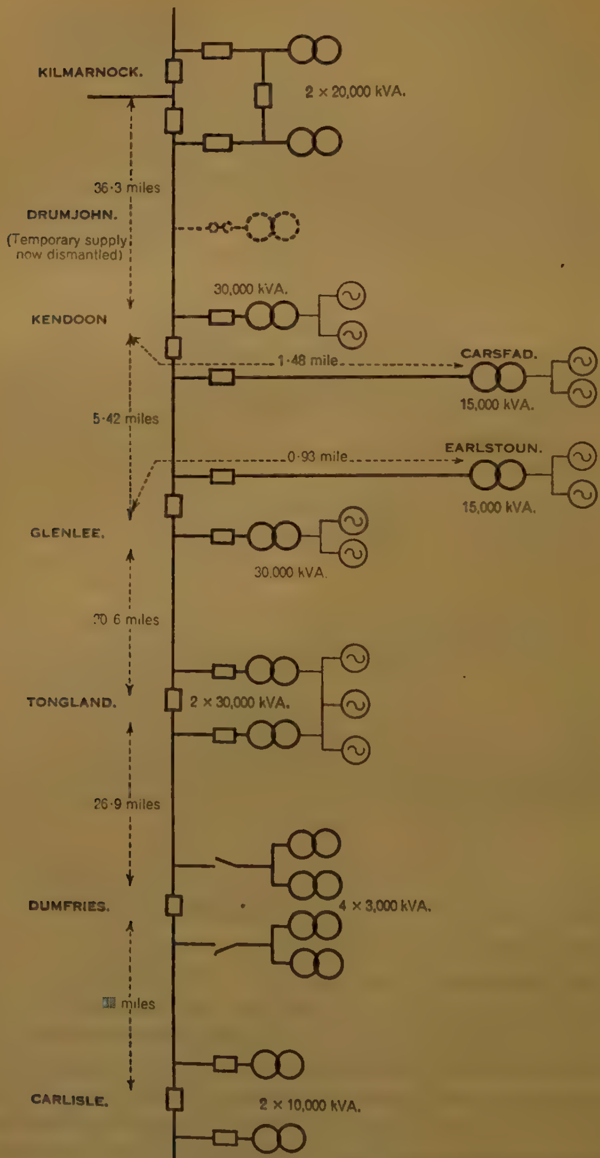


Diagram of the 132-Kilovolt Equipment in South-West Scotland.

of generating plant no longer have to be built as near to the centres of distribution-systems as they were before the Grid was established. The capacity of the 132-kilovolt single-circuit transmission-line was considered to be of the order of 50,000 kilowatts on the assumption that the length of the line would not exceed 50 miles, and this capacity was governed by considerations of voltage-regulation.

It was, however, foreseen that the Grid would open up new possibilities of utilizing water-power schemes, and the capacity of the transmission-lines would have to be examined in each instance. The simplest and cheapest method of connecting the Galloway scheme with the existing 132-kilovolt lines in Central Scotland and in North-West England was by means of a single-circuit line between Kilmarnock and Carlisle. The length of the equivalent single-circuit 132-kilovolt line from the Galloway generating stations to the receiving substations in Central Scotland is about 80 miles, and that to the receiving stations in North-West England is about 160 miles.

The standard design of single-circuit 132-kilovolt transmission-line adopted by the Central Electricity Board has the following electrical characteristics: a resistance of 0·25 ohm, a reactance of 0·67 ohm, and a charging current of 0·33 ampere per mile per phase at 132 kilovolts and 50 cycles. The 132-kilovolt transformers of the standard design adopted by the Central Electricity Board have an impedance of about 10 per cent., and are fitted with on-load tap-changing gear which enables the ratio of transformation to be varied between the limits of $\pm$ 10 per cent. It is of interest to note that a 30,000-kilovolt-ampere transformer has an impedance-voltage of 7·6 kilovolts when operating at its full load of 132 amperes; the impedance per phase is therefore 57·5 ohms, which is equivalent to that of 82 miles of single-circuit transmission-line.

The full-load voltage-drop in the 132-kilovolt transformers at a power-factor of 95 per cent. is of the order of 3 per cent., and allowing for transmitting and receiving transformers the total voltage-drop through the transformers should not exceed 6 per cent. As the on-load tap-changing gear allows the ratio of transformation to be varied by 20 per cent., the voltage-drop for the transmission-lines should not exceed 14 per cent.

Before considering the actual problem of voltage-drop over the transmission-lines, it should be remembered that the charging current of the lines amounts to 75 kilovars per mile at 132 kilovolts, and it can be assumed without appreciable error that this charging current is concentrated at the two ends of the line under consideration. Under full-load conditions, when the demand for reactive kilovolt-amperes is high, this charging current is an advantage because it has the effect of reducing the reactive kilovolt-amperes which have to

be transmitted. The magnetizing current of the transformers, however, tends to offset the charging current of the line, and in those parts of the Grid where the transforming stations are close together and where large 132-kilovolt transformers are installed the magnetizing current is about equal to the charging current. On longer lengths of line, such as those connecting the Galloway scheme with the Central Scotland and with the North-West England Schemes, the magnetizing current is about one-fifth of the line-charging current.

The general requirement, therefore, is to find within what limits of

TABLE XVII.

Example.	1	2	3	4
Distance: miles . .	160	80	160	160
Kilovolts at sending end	140	140	140	147·5
Amperes at sending end	235	235	117·5	235
Kilovolt-amperes at sending end . .	57,000	57,000	28,500	60,000
Power-factor at sending end: per cent .	99·3	93·5	93·5	94·4
Kilowatts sent out .	56,600	53,300	26,650	56,600
" lost . .	6,600	3,300	1,650	6,600
" received .	50,000	50,000	25,000	50,000
Kilovars sent out. .	6,700	20,200	10,100	19,900
" lost . .	17,600	8,800	4,400	17,600
" received .	−10,900	11,400	5,700	2,300
Kilovolt-amperes received	51,120	51,270	25,630	50,050
Kilovolts at receiving end	125·6	125·9	125·9	123·0
Kilovolt-amperes lost	5,880	5,730	2,870	9,950
Voltage-drop: per cent.	10·3	10·1	10·1	16·6
Power-factor at receiving end: per cent.	97·8 leading.	97·5 lagging.	97·5 lagging.	99·9 lagging.
Efficiency: per cent..	90	95	95	90

power-factor 50,000-kilowatt loads can be transmitted, and to establish definite relationships between the regulation of the line and the load transmitted. Table XVII gives four examples of the transmission of load over distances of 80 and 160 miles of equivalent single-circuit 132-kilovolt line.

Example 1 of Table XVII shows the regulation for the reception of 50,000 kilowatts of load at a leading power-factor of 97·8 per cent. when transmitted over a distance of 160 miles. Examples 2 and 3 show that if the power-factor of reception is altered from 97·8 per

cent. leading to 97·5 per cent. lagging, then for the same voltage-drop either the load or the distance of transmission must be halved. Example 4 shows the effect of receiving a load of 50,000 kilowatts at a power-factor of 99·9 per cent. lagging when transmitted over a distance of 160 miles, and this can be taken as approaching the limit to which transmission over the single-circuit 132-kilovolt line can conveniently be carried. The Author is indebted to his friend, Mr. C. G. Carrothers, for the method of calculating and setting-out the above results, and for a simple method of analysis of regulation-problems which is given in the Appendix (p. 421).

The above results show the extent to which it is possible to transmit loads over single-circuit 132-kilovolt transmission-lines. In any specific instance there are other operating conditions which may affect the voltages both of the sending and of the receiving stations. It is assumed that the normal operating condition would be for the Galloway stations to be connected in parallel both with Central Scotland and with North-West England, and the operating voltages of the Grid lines in Central Scotland, for instance, are affected by the import of load from the hydro-electric generating stations in the Grampians. It is fortunate that in an interconnected system the power-factor of load-transmission adjusts itself to comply with the conditions of voltage maintained at the ends of the transmission-lines. If the transmission-line from Galloway to Central Scotland or to North-West England should be out of service, there is less restriction on the transmitting voltage of the remaining line, and some increase in the load which can be transmitted is then possible. The transmission-lines, therefore, act as a partial standby to each other in providing an outlet for the power generated in the Galloway hydro-electric station. The limiting feature may then be the current-density in the conductors rather than the voltage-regulation.

The conclusion was reached that a 132-kilovolt single-circuit transmission-line of the standard construction of the Central Electricity Board could deal with the transfer of the loads from the Galloway scheme if reasonable use were made of the facilities for control given by the standard range of 132-kilovolt transformer tap-changing gear adopted by the Central Electricity Board, and a high power-factor of transmission were maintained by generating reactive kilovolt-amperes in suitable proportions at the various generating stations. It was decided to give the control engineer in Central Scotland a continuous indication of the output of some and of all the generating stations of the Galloway scheme, and of the proportion of the output transmitted to Central Scotland and to North-West England.

The Equipment of the Central Electricity Board forming the Interconnexion between the Galloway Scheme, Central Scotland, and North-West England.

The design of the 132-kilovolt transmission-system of the Central Scotland Electricity Scheme was described by the Author in 1931.[1] That Scheme, which was the first of the regional schemes of the Central Electricity Board, was adopted in 1927. The South Scotland Scheme, which was the last of their regional schemes, was adopted in 1931, and the Author proposes to refer only to the changes in the design of the equipment during this intervening period, and to describe any special designs called for by the particular requirements of the South Scotland Scheme.

The general layout of the single-circuit 132-kilovolt transmission-line between Kilmarnock and Carlisle was chosen to avoid double-circuit lines leading to any individual generating station of the Galloway scheme. It was considered that it would be unwise to allow the possibility of any mechanical damage to one double-circuit tower interrupting the complete output of any of the generating stations. It must be admitted that the double-circuit sections from Kendoon to the Carsfad station and from Glenlee to the Earlstoun station do not fulfil this requirement, but in view of the saving in capital expenditure by the use of these double-circuit lines, and in view of the relatively small output of the stations at Carsfad and Earlstoun, it was decided in this instance to adopt this method of construction.

There were no major modifications in design of the 132-kilovolt transmission-lines during the period under review, but the following small variations in design may be of interest. The design of bolted clamp used to connect the line-conductor to the tension-insulator sets is of the snail type in South Scotland, as compared with the parallel-plated bolted type in Central Scotland. The route of the transmission-lines in South Scotland was largely in rocky and stony ground, and for this reason the concrete-ball type of tower-foundation (in which the cavity in the ground was formed by exploding a charge of dynamite) and the earth-grillage type of foundation were not used, and all the foundations were of the concrete type. It was decided to increase the factor of safety of the transmission-line where it passes over high ground between Kendoon and Kilmarnock, and the factors of safety, which in general are 2·5 on the elastic limit of the tower-members, 2·5 on the electro-mechanical test of the insulator-sets, and

[1] "The 132-kilovolt Transmission-System of the Central Scotland Electricity Scheme." Minutes of Proceedings Inst. C.E., vol. 232 (1930—31, Part II), p. 245.

2·2 for the line-conductor, were increased by 20 per cent. where the line is more than 700 feet above sea-level.

The layout of 132-kilovolt switchgear had developed during the period under review, and in general the mesh-connected system was adopted. The low type of concrete structure was used, and *Fig. 46* (p. 410) indicates the various types of substation between Kilmarnock and Carlisle. There is a one-switch substation at Dumfries which can be extended in the future to a three-switch substation. Tongland, Glenlee, and Kendoon are three-switch substations, and Kilmarnock is a three-switch substation which has already been extended to a five-switch substation.

Tongland switchgear is of special design, and is the only example of completely metal-enclosed 132-kilovolt switchgear in Great Britain. The connexions between the terminal towers and the line-isolators and between the transformer-isolators and the windings of the transformers are by means of oil-filled 132-kilovolt cable. The isolators adjacent to the circuit-breakers are of the vertical piston type, in which the inner insulated conductor forms a piston sliding axially within a cylinder filled with insulating oil.

The design of 132-kilovolt switchgear was improved during the period under review, largely due to the results of tests at short-circuit testing stations, and the plain explosion-pot design of circuit-breaker was superseded by oil-blast pots in which arc-control devices are incorporated. Tests show that the time taken to interrupt the arc at the rated short-circuit of 1,500,000 kilovolt-amperes is from 12 to 14 cycles with the plain explosion-pot, but only about 6 cycles with the oil-blast pot. The operating mechanisms were also designed to obtain more rapid operation.

The grouping of the 132-kilovolt switchgear at substations adjacent to three of the five hydro-electric generating stations was not allowed to affect the position of the 132-kilovolt transformers which are placed adjacent to each of the generating stations. It was decided, however, to reduce the amount of standby transformer-capacity at Glenlee, Earlstoun, Carsfad, and Kendoon, in view of the relatively small size of the generating stations. This was possible because an 11-kilovolt transmission-line was provided by the Galloway Power Company to give a general supply of electricity for local requirements, particularly during the times when the generating stations are shut down.

In Central Scotland, 10,000 kilovolt-amperes was the smallest size of 132-kilovolt transformer supplied, but in South Scotland 3,000-kilovolt-ampere transformers were required. The on-load tap-changing gear of transformers of a smaller size than 10,000 kilovolt-amperes forms part of the low-voltage winding instead of the 132-

kilovolt winding. The 3,000-kilovolt-ampere transformer at Glenlee was protected by carbon-tetrachloride glass-enclosed fuses, which were the first 132-kilovolt fuses to be used in Great Britain.

The 30,000-kilovolt-ampere transformers at Tongland are of non-resonating design, in which the 132-kilovolt windings are shielded to form an alternative path for charging currents. The graded shields are connected to the line end of the 132-kilovolt windings to make the distribution of voltage under impulse conditions as uniform as under the normal 50-cycle conditions. These transformers were designed for an impulse-test of 700 kilovolts, and an induced-voltage test of 280 kilovolts at 50 cycles. The practice of not installing lightning-arresters on the 132-kilovolt lines was continued, but the magnitude of surges which may reach the Grid transformers has now been controlled by fitting adjustable co-ordinating gaps, the horns of which have been set at 28 inches apart.

The subsidiary equipment was very similar to that used in Central Scotland. Similar oil-filtering plant and storage-plant was provided for each of the three-switch substations, but oil-storage plant only was provided at Dumfries. A portable oil-filtering plant is available for this substation, and for the transformer equipments at Carsfad and at Earlstoun. The system of metering equipment was similar to that used in Central Scotland. Impedance protective gear was used for long sections of the 132-kilovolt transmission-lines, but at Glenlee and at Kendoon, where only one transformer is installed, the potential-reference for the impedance-relays has been obtained by three single-phase non-resonating type 132-kilovolt potential-transformers connected direct to the lines. A fourth single-phase potential-transformer was provided at these sub-stations on the other incoming line to enable the 132-kilovolt section switch to be used for synchronizing the Central Scotland system with the North-West England system.

A telephone- and pilot-cable was suspended on a catenary wire from the 11-kilovolt wood-pole transmission-line of the Galloway Power Company between Glenlee and Kendoon, and this is used jointly by the Galloway Power Company and the Central Electricity Board. Special cores were provided in this cable for an opposed-voltage type of protective gear for the section of 132-kilovolt line which runs direct from Glenlee to Kendoon. Additional cores are used for the restricted-earth-leakage protective gear which covers the 132-kilovolt windings of the Earlstoun and Carsfad transformers and the 132-kilovolt spur lines between Glenlee and Earlstoun and between Kendoon and Carsfad. Negative-phase-sequence protective gear had to be provided at the substations adjacent to the one-switch substation at Dumfries to ensure the clearance of any fault in the

transformers at Dumfries, and, in addition, intertripping circuits have been provided which make use of the supervisory-control equipment for the transmission of the necessary signals.

It was decided that the central indicating and load-dispatching station installed in Glasgow for the Central Scotland Scheme should be extended to deal with the South Scotland Scheme, but the distance of Galloway from the Post Office trunk lines made it necessary to design the equipment on the tandem system instead of on the more general radial system. Advantage was taken of the underground trunk line of the Post Office between Glasgow, Dumfries, Carlisle and Manchester to use a special four-wire circuit designed for particularly low losses ; this is a repeater circuit, and voice-frequency equipment had to be used. The circuit between Glasgow and Dumfries gives the usual facilities for speech between the central indicating station and the various substations and hydro-electric generating stations, and for the usual switch and tap-changing gear position-indications from the various switching and transforming stations. The pilot cable of the Galloway Power Company has been used for speech and other indications between Glenlee, Earlstoun, Carsfad and Kendoon.

The special circuit in the Carlisle–Manchester cable of the Post Office, referred to above, is also used to give a continuous indication at the central indicating stations in Glasgow and Manchester of the kilowatts and kilovars transferred from the Galloway scheme to North-West England, and a system of continuous metering indication of the output of the Galloway scheme and of the way in which this output is transferred to Central Scotland and to North-West England is provided at the central indicating station in Glasgow.

A system of photo-telemetering was developed to give these remote indications. At the originating end it consists of an optical system to convert the readings of the indicating kilowatt- and kilovar-meters into impulses of voice-frequency currents, which are then transmitted over the Post Office circuits. At the receiving end the impulses are modified and passed into moving-coil instruments which are scaled to give similar readings to those at the originating end of the line. It is important that the transmission of these readings should be accurate, and the system does not depend on the magnitude of the voice-frequency impulses, but on their relative durations : the transmission is therefore independent of variations in the characteristics of the telephone-circuit.

An analysis of the cost of the equipment of the Central Electricity Board in South Scotland is given in Table XVIII (p. 418). The unit costs of the equipment are very similar indeed to those for the equipment in Central Scotland.[1]

[1] *Loc. cit.*

TABLE XVIII.—ANALYSIS OF COST OF CENTRAL ELECTRICITY BOARD EQUIPMENT.

132-kilovolt transmission-line	£1,530 per circuit-mile.
132-kilovolt equipment :—	
Foundations	£6,200 per three-switch substation.
Switchgear	£6,220 per switch for three-switch substations.
Transformers : 3,000 kilovolt-amperes	£0·885 per kilovolt-ampere capacity.
15,000 „	£0·500 per kilovolt-ampere capacity.
30,000 „	£0·32 per kilovolt-ampere capacity.
11-kilovolt and other equipment :—	
Main cable-connexions and 11-kilovolt switchgear	£6,270 per three-switch substation.
Metering, remote-indicating and other incidental equipment	£4,900 per three-switch substation.

The South Scotland Electricity Scheme was adopted in August 1931, and special steps were taken to accelerate the construction of the 132-kilovolt transmission-line from Kilmarnock to Glenlee and to Tongland, to make electricity available for the contractors of the Galloway Power Company. This supply was given at Glenlee in July 1932 and at Tongland in October 1932. A public supply was available for the Kirkcudbright County Council at the end of December 1932. The interconnexion with North-West England at Carlisle was completed in March 1933. The 132-kilovolt fuse and 3,000-kilovolt-ampere transformer equipment was transferred from Glenlee and installed at a direct tapping from the 132-kilovolt line at Drumjohn in August 1934 to make electricity available to the contractors of the Galloway Power Company in the neighbourhood of loch Doon. The permanent transformer-equipments at Glenlee and at Tongland were put into service in the autumn of 1934, and the transformer-equipments at Earlstoun, Carsfad and Kendoon in the autumn of 1936.

During the operation of the scheme there have been no major troubles on the electrical equipment of the transmission-system. There were, however, some troubles due to snow deposits on the 132-kilovolt line during the blizzard of February 1933; there have also been some faults due to lightning, which appears to be rather prevalent in the Galloway district.

The main 132-kilovolt interconnecting equipment was manufactured and erected by the following Contractors: transmission-lines by Messrs. British Insulated Cables Ltd.; metal-enclosed switchgear by Messrs. Reyrolle & Co. Ltd.; open-type switchgear by Messrs. The British Thomson-Houston Co., Ltd.; 30,000-kilovolt-ampere

non-resonating transformers by Messrs. The British Thomson-Houston Co., Ltd.; 15,000-kilovolt-ampere transformers by Messrs. Bruce Peebles & Co., Ltd.; and 3,000-kilovolt-ampere transformers by Messrs. Metropolitan-Vickers Electrical Co., Ltd.

General Considerations.

The average annual load-factor of generation in Great Britain is of the order of 34 per cent., and the Galloway hydro-electric scheme with an average annual load-factor of 20 per cent. comes into the category of peak-load plant. It is natural to compare the cost of generation of electricity by such peak-load plant with the cost of generation by the most modern steam generating stations, particularly if the cost of coal continues to rise.

Existing steam generating stations in Great Britain have not been designed specially to deal with peak loads, but at the present time the total generating cost of electricity from a modern steam generating station, designed to deal with the general supply of electricity, amounts to about 0.2*d.* per unit sent out when operated as a base-load station with an annual load-factor of the order of 60 per cent. This cost is increased to about 0·425*d.* per unit for the same generating station if it is operated as a peak-load station with an annual load-factor of 20 per cent.

It is found that the total capital charges and working costs of the Galloway hydro-electric scheme are a little under £240,000 per annum, which with an output of 180 million units per annum corresponds to a total cost of generation of electricity of 0·32*d.* per unit. It will be seen therefore that, without making any allowance for spare plant or for the cost of any additional interconnecting equipment, the actual total generating cost of electricity from the Galloway scheme is about 75 per cent. of the equivalent cost from the most modern steam generating stations. This comparison is only valid if water is available to operate the hydro-electric plant to its maximum capacity at the times of peak load on the system.

The Author suggests that a re-examination of water-power resources to establish the estimated cost of the generation of electricity at various load-factors would be of value to enable a comparison to be made with the cost of generation from steam stations. The capability of the single-circuit 132-kilovolt transmission-lines of the Central Electricity Board to deal with the transmission over long distances of loads of the order of 50,000 kilowatts has been demonstrated, and it should be pointed out that the capacity of the existing transmission-lines could be increased if it were found to be necessary by the installation of synchronous condensers at suitable positions.

The Author wishes to thank Mr. Johnstone Wright, M. Inst. C.E., Chief Engineer of the Central Electricity Board, for permission to present this Paper, and is indebted to Mr. S. B. Donkin, President Inst. C.E., senior partner of Messrs. Kennedy & Donkin, for his encouragement and advice during the preparation of the Paper.

The Paper is accompanied by two sheets of drawings, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Transmission over 132-Kilovolt Lines.

The figures given in Table XVII (p. 412) apply to a line without capacity. The effect of the line-capacity on the transmission can be obtained with sufficient accuracy by replacing the total distributed capacity by a condenser of half that capacity at each end of the line.

The figures given in example 4 of Table XVII, for instance, can be adjusted to take account of capacity as follows. The charging current per mile is 75 kilovolt-amperes at 132 kilovolts. A condenser with a charging current equal to that of 80 miles of line (6,000 kilovolt-amperes at 132 kilovolts) should be considered as connected at each end. The charging kilovars vary with the square of the voltage. Then :—

Kilovars sent out (from Table XVII, example 4) = 19,900 ;
Kilovars contributed by half line-capacity at sending-end voltage of 147·5 kilovolts = 7,490 ;
hence difference to be delivered to line by transformers at sending end = 12,410 kilovars.
The power-factor at the sending end is improved from 94·4 per cent. (Table XVII, example 4) to 97·6 per cent. lagging.
Kilovars received (from Table XVII, example 4) = 2,300 ;
kilovars contributed by half line-capacity at receiving-end voltage of 123 kilovolts = 5,100 ;
hence total delivered from line to transformers at receiving end = 7,400 kilovars.
The power-factor of total load delivered from line to receiving end is reduced from 99·9 per cent. (Table XVII, example 4) to 99·0 per cent. lagging.

The capacity of the line is thus seen to make an appreciable contribution to the received current.

Strictly speaking, the line with distributed capacity should be replaced by a line with a reduced impedance and an increased capacity. The amounts of the adjustments are small, as shown by the following examples.

Length of 132-kilovolt line : miles.	Corrections for distributed capacity.		
	Reduction in impedance : per cent.	Phase-shift of impedance : degrees.	Increase in capacity : per cent.
100	0·5	+0·13	0·2
200	1·8	+0·42	0·97
300	4·2	+0·94	2·2

The percentage-corrections are well within the errors in other assumptions which have to be made, up to lengths of 200 miles.

The replacement of the distributed capacity by condensers at the ends of the

line allows the current to be considered as constant from end to end of the line. On this assumption the voltages at the sending end and receiving end are exactly proportional to the kilovolt-amperes at the sending end and the receiving end.

To calculate the change in kilovolt-amperes from end to end of the line, the power losses I^2R and reactive losses I^2X are subtracted from the power and reactive loads respectively at the sending end.

The formula for receiving-end 3-phase star voltage V_R with a sending-end star voltage V_S and current I at power-factor $\cos\phi$ is usually written

$$V_R = \sqrt{(V_S \cos\phi - RI)^2 + (V_S \sin\phi - XI)^2}.$$

The method of calculation by losses amounts to multiplying and dividing both sides of the equation by $3I$, and obtaining

$$\frac{3\,V_R\,I}{3I} = \sqrt{\frac{(3V_S\,I\cos\phi - 3RI^2)^2 + (3V_S\,I\sin\phi - 3XI^2)^2}{3I}}.$$

In this equation the first pair of brackets under the root contains the total power sent out, $3V_S\,I\cos\phi$, reduced by $3RI^2$, the total power-losses in the three phases. The second pair of brackets under the root contains the total reactive load sent out, $3V_S\,I\sin\phi$, reduced by $3XI^2$, the total reactive losses in the three phases. By obtaining the relationship between the sending and receiving voltages in this way, the phase-swing and line-losses are derived in the process of making the calculation.

Discussion.

Mr. Hunter. Mr. J. K. Hunter exhibited a number of lantern-slides illustrating e Papers; models of two of the power-stations and of the complete arlstoun works were also shown.

Mr. Williams. Mr. F. H. Williams exhibited a cinematograph film illustrating e construction and operation of the works, the behaviour of some the hydraulic structures and equipment under flood-conditions, d the working of the fish-passes.

The President. The President congratulated the Authors on the most compre-ensive and interesting set of Papers that they had presented. The heme was one which was economically possible only because of e Central Electricity Board's Grid, and only if used for peak-load urposes. In discussing the question of peak-load stations, it should e borne in mind that the operating costs of a peak-load station ould be higher than those of an ordinary station provided that the pital charges were correspondingly lower.

He would mention that the line-faults due to lightning referred to pp. 400 and 418 had been only on the 11-kilovolt lines belonging the Galloway Power Company, and the 132-kilovolt lines of the entral Electricity Board did not suffer in that way.

Messrs. Hawthorne and Williams stated on p. 405 that the overall ost was £28·3 per kilowatt of power available. It would be seen om p. 392, however, that the scheme frequently delivered slightly ore than the nominal output of the installed plant. It was an teresting economic point that a station or group of stations of the ind in question could be worked to its full installed capacity, earing in mind that the stations were working only at about 20 per ent. load-factor, the diversity being between summer and winter. he scheme was the only system of that type working in Great ritain.

Mr. Mountain stated that the total generating costs of the Galloway cheme were less than would be possible with standard steam enerating stations. That was, of course, perfectly true, but it would e fair to add that a specially-constructed steam peak-load station ould work at a total cost equal to or possibly even lower than that f the Galloway scheme. He agreed with Mr. Mountain's suggestion hat a fresh examination should be made of all possible water-power

The President. developments in Great Britain, in order to compare the total cos of generation by hydro-electric and steam plants, including capit costs, working costs, and, in the former case, transmission cos Such a survey would be of particular value now that the costs fuel and labour were continually increasing.

Lord Meston. The Right Hon. Lord MESTON expressed his warm appreciati of the compliment that The Institution had paid him by inviti him to join in the Discussion. He accepted that courtesy mainly order to offer a respectful tribute on the part of himself and h colleagues on the directorate of the Galloway Water Power Com pany to the skill, the resource, the patience and the courage of th engineers who had contributed to the construction and completi of the fine work described in the Papers.

That work took advantage of two great drainage-systems an provided energy for the industries of the country from Glasgow the north down to Carlisle in the south, without in any appreciab degree affecting the amenities of a beautiful and unspoilt part the country. The dams, with their graceful curves, fitted almo harmoniously in with the folds and contours of the surrounding hill The power-houses, originally a little grim, were adapting themselv slowly but surely to the landscape, and even the surge-towers, whic were the most intractable of all the buildings from that point of view were gradually merging into the picture. If any captious observ were to criticize the engineers for any shortcomings of their work i that respect, it should in fairness be pointed out that that wa far more than counterbalanced by the new beauty which the had introduced into the countryside. They had enriched the lan with a whole series of new lakes, which in their beauty, the size, and the variety of their shape and their surroundings, wer a striking addition to the beauty and attractiveness of the land o Galloway.

As an outside observer, he had been very vividly impressed b two special problems which the engineers had taken in their stride firstly, the geology of the country, and, secondly, the fish of th waters. Most engineers would be familiar enough with bad roc and with tricky foundations, but if they wanted to know ho deceptive appearances might be they should go to Galloway an see for themselves. A bank of apparently beautiful granite, firm and solid, would seem to be an ideal foundation, but, as often as no when excavation was begun it would prove to be simply a she overlying a whole stratum of shattered rock and debris, little bett than loose rubble. One Paper described how disastrous that migh have been at Tongland; the same trouble, however, was alway rising up against the engineers at almost every point, and mus

ve added enormously to their anxiety, as it did to the difficulty **Lord Meston.** d the costliness of the work.

The problem of satisfying the requirements of the fishing interests d called forth another quality in the engineers concerned, that of plomacy. It had required all their diplomacy to reconcile the sidents and the neighbourhood generally to the future of the lmon, and to reconcile the salmon to the arrangements made for em; but, as would have been seen from the film that had been own, that had been successfully achieved.

He would like to add a word of personal memory and tribute, to y how immensely all who knew him had admired and how deeply ey all lamented the late Colonel McLellan, whose initiative, triotism and almost prophetic foresight had been so largely sponsible for the work described. Colonel McLellan gave some the best years of his later life to the task of imagining, conceiving, d directing the construction of the great work; his friends d decided to put up a small commemorative plaque on the wall the Tongland power-station, but the whole scheme was his onument and would keep him fresh for ever in the memory of his iends.

Mr. W. J. E. Binnie, Vice-President, observed that Messrs. Hudson **Mr. Binnie** d Hunter, dealing with the question of flood-discharge, stated on p. 7 that a discharge of 21,800 cubic feet per second had been recorded Earlstoun, which was larger than that obtained from the curve r normal maximum floods recommended by the Institution Comittee on Floods in Relation to Reservoir Practice.[1] He would like say a word about that Committee. It had been set up under his airmanship, shortly after the passage of the Reservoirs (Safety) ct, to investigate the possible intensity of floods in Great Britain, as ie of the chief causes of failure of reservoirs in the past had been the sufficient allowance which had been made for the disposal of floodater, and as there was considerable divergence of opinion among gineers as to what intensity should be allowed under different nditions. The total flood mentioned on p. 327, including the antity which was being diverted to loch Doon, had been 22,300 secs, which was 14 per cent. more than that which was given y the curve attached to the Report of the Committee. That rve was, of course, supposed to deal with normal maximum ods, and allowance had to be made for the particular contions which applied in any one place. The flood-discharge, for stance, would obviously be more rapid and would produce a gher peak if the inclination of the ground were very steep. The

[1] Footnote (1), p. 330.

Mr. Binnie. general criticism levelled at the Report, however, had been th discharges estimated from that curve were too high, so that loc authorities were being put to unnecessary expense in putting th house in order, now that the Report had been adopted by ma engineers when reporting to their clients under the recent legislatio He was very glad, therefore, to find in the present case a dischar which exceeded that given by the curve. He took the opportuni of asking for information with regard to flood-discharges fro anyone present who had had the opportunity of gauging a floo of great intensity. The Report of the Flood Committee was on an interim one, based on very scanty data, and it was intended publish a better report later when the necessary information becar available. For the Committee's information, Sir Alexand Gibb had supplied particulars of the flood at Glenlochar 1872; that flood had been 10 per cent. below the Committee curve. No one would pretend to measure a flood-dischar with any great degree of accuracy, and if divergences fro the normal curve were not more than 10 to 14 per cent., l thought that that was quite satisfactory. He would like to ask th Authors how the flood-discharges referred to in the Paper had bee measured. It was curious that the balancing effect of loch Ke had had the effect of reducing the flood at Glenlochar to about per cent. of what it was at Earlstoun, although the top water ar of the reservoir itself, which was all-important, was only 1 p cent. of the drainage-area. Had the loch been drawn down at a before the flood, or was it already overflowing when the floo reached it?

The compensation-water required for the river Doon, equivaler to an average rate of 48 million gallons per day, was very excessiv On p. 331 that rate was stated to correspond to "a rainfall 24·0 inches over the catchment, or 33 per cent. of the averag annual rainfall." Parliament very seldom awarded more con pensation-water than one-third of the average run-off during th 3 driest consecutive years, and if that basis had been adopte in the present case the compensation-water would have been equ valent to only 14 inches instead of 24 inches. He would like t direct attention to the serious position that might arise in a dı year; there might be only two-thirds of the average rainfall, an if it were necessary to maintain a flow equivalent to 24 inch down the river Doon there would be only 10 inches available fo power-supply, assuming an evaporation-loss of 14 inches. I that case the storage of loch Doon, which was 2,930 million cub feet, would only be utilizable for power purposes to the extent about 860 million cubic feet.

A great enemy of such a scheme as that described was frost, and Mr. Binnie. ost unfortunately occurred during the period when the peak emand might be expected.

Mr. RUSTAT BLAKE expressed the regret of his partners and himself Mr. Blake. hat their senior partner, Sir Alexander Gibb, Past-President nst. C.E., was unable to be present on account of illness. From he very beginning of the works described, Sir Alexander had aken the liveliest interest in the whole project and had followed s progress very closely until he was able to hand it over to the ompany.

Messrs. Hudson and Hunter had referred to the work of the menities Committee. The help which that Committee gave had een extremely valuable in many cases where difficulty might otherwise have been experienced, and he believed that the Committee as satisfied that the engineers had done all that they could to carry ut its wishes.

It was of the greatest value that the three Papers had been resented as one article. Their grouping had delayed their production somewhat, but it had allowed of a particularly valuable contribution in the Paper by Messrs. Hawthorne and Williams giving the esults of a full year's working of the whole system. The figures iven were certainly most satisfactory.

With regard to the fish-passes, he had to admit that when he first aw the Tongland fish-ladder, before the water was turned into it, e wondered whether fish would ever really take to it. However, Mr. Hudson, who was watching on the second day after the ladder ad been put into use, reported that there were fish in the second pool, nd in fact the fish took to it readily. A little over a week later there as a certain amount of trouble with the fishing proprietors, and the ngineers were so satisfied that all was well that they invited them o come and see what was happening. They came on a Sunday; he sluices were closed down, and a count showed that 64 salmon ere on their way up the ladder, which, he thought, was remarkably good considering the short period during which it had been perating.

The section of the Paper by Messrs. Hudson and Hunter dealing ith the manufacture of the concrete was very interesting. He was ery glad that that section had been included; it might be thought ather lengthy, but it should be borne in mind that the cost of the oncrete placed amounted to about 43 per cent. of the whole cost of he constructional works, which showed how largely concrete figured herein. Having studied the concrete from every point of view, in unnels, dams and so on, he was firmly convinced that for work of the ind in question it was very dangerous to employ highly quick-setting

Mr. Blake. and very finely-ground cements. Further, if it were desired to secu good concrete and a good finish in water-tunnels, good pit sand shou be used. It was undesirable to rely on crushed rock, howev good the rock might be, because when the work was stripped a gre deal of repair-work would have to be done, which ought not to necessary.

Mr. Hobson. Mr. Harold Hobson remarked that economic considerations, the Authors pointed out, dictated that the works should be design for peak-load operation. The particular scheme described, howev was really intermediate in nature between a base-load station and true peak-load plant required for operation during a few hours of t day and a few months of the year only. Peak load was always important problem in electricity-supply, and the difficulty of pro viding economically for the generation of power required ov comparatively short periods was one which was always before tho responsible for the economic production of electricity in Gre Britain.

He did not entirely agree with the opinion expressed by Mess Hawthorne and Williams on p. 377 that an increase in the intensi of the peak was likely to follow from a more intensive developme of domestic load. That might be true of a small undertaking, b when dealing as in the present instance with a large part, or inde with the whole, of Great Britain, he thought that the development domestic load was quite as likely to fill in the valleys as to increa the peak. Again, the Authors pointed out that the economic provision of peak power required primarily low capital cost; it d not necessarily require high efficiency or low fuel-consumptic The greater part of the peak-load of Great Britain was at prese being dealt with by the older and less efficient of the existi generating stations, and he thought that for the most part, as t President had hinted in his remarks, it would continue to be de with by those stations, reconstructed and extended as might necessary. Galloway was not a scheme that had an inherent low capital cost; as was pointed out in the Papers, the capital co was about twice that of an ordinary steam station, but the operati costs were exceptionally low. It hardly fulfilled, therefore, t requirements of a full peak-load station, and for that very reason had been designed to take an intermediate position, operating approximately 20 per cent. load-factor.

It had been stated that the scheme had been made possible by t transmission-lines of the Grid being available to carry away t output, but it should be pointed out that the reverse was equa true. It would not have been commercially practicable to constru that section of the Grid required to connect the industrial areas

entral Scotland with those in Lancashire had not the Galloway cheme provided a paying load to transmit over the Grid. Incidentlly, the scheme had made possible the extensive developments of ural supply in the counties of Kirkcudbright and Dumfries. **Mr. Hobson.**

Before its adoption the scheme had been considered very fully y the Central Electricity Board in comparison with the alternative f extending steam generating stations in the neighbouring industrial reas, and strong representations had been made to the Board by the oal-mining interests, who held that on balance the advantage lay ith coal-fired stations rather than with the proposed development. t was true that at that time the margin in favour of the Galloway cheme had not been very large, but the action that had been taken ecently by the coalmasters had certainly proved their arguments gainst the Galloway scheme to be unsound, for the price of coal ad gone up by 48 per cent. in Central Scotland and by 22 per cent. n North-West England, since the estimates had been made ; there ould now be no doubt at all that the scheme was a thoroughly sound ne in comparison with coal-fired production.

Finally, he would like to say a word of appreciation of the very aluable co-operation that the Galloway Company's staff had given o the Central Electricity Board in carrying out the necessarily omplex operations which were required to meet the varying demands f the load in the two adjoining areas.

Mr. E. Bruce Ball remarked that very many interesting and ovel applications of water-control apparatus had been introduced in he Galloway works, and he would like to pay a tribute to the engineers esponsible for the scheme for their attitude in regard to the adoption f advanced designs. He would like to mention a few of the most nteresting examples. **Mr. Ball.**

The Glenlochar barrage incorporated a number of details which, t the time of its construction, departed from hitherto accepted ractice. The pleasing appearance and the apparent lightness of he structure were largely due to the elimination of the accepted orm of balance-box by making the counterweights of cast-iron and ousing them within the grooves in the piers, covered on the outside y steel plates flush with the surface of the concrete. The adoption f that form of counterbalance made it inconvenient to use wire oisting ropes with the conventional type of winding-drum, and ifting-chains of the bush-link roller type, attached at the ends to the ate and counterweights respectively, were adopted. The power vas transmitted to those chains through machine-cut sprocketwheels driven from the central headstock and horizontal shafts. That arrangement was also followed in the case of the flood-gates t Earlstoun and Tongland, except that the piers, being larger, were

Mr. Ball.

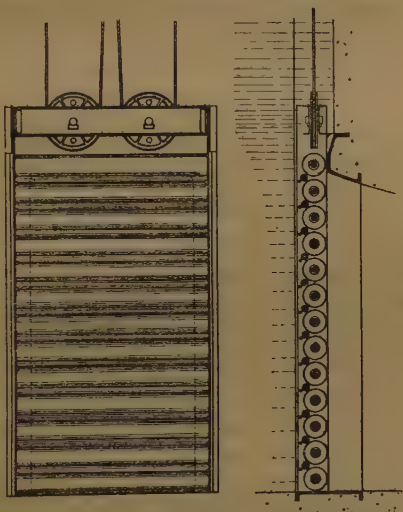
made hollow to house the counterweights, and steel cover-plat were thus unnecessary. Some indication of the arrangement wa given in *Fig. 15* (p. 345), which showed a section through the Tonglan gates. Since the gates were arranged for alternative manual oper tion, it was desirable to protect the operator from harm when usin the crank-handle, should the remote control at Tongland be operate A device for that purpose was incorporated in each headstock, an comprised a hinged cover closing a casing which surrounded t square end of the shaft on which the crank-handle was fitted, t cover incorporating an electrical switch in the hinge, and isolatin the motor-contactor control circuit when it was open. Th circuit could not be restored until the crank was removed and t cover completely closed. It was significant that the above-mention novel features had been applied with success to many subseque sluice-gate structures.

The limited time available for the completion of the barra necessitated the removal of the cofferdam immediately each grou of piers was completed. That precluded the assembly and rivetin of the gates on their sills, as would normally be done ; each gate wa therefore transported by road as a completely-assembled unit fro the shops to the site. A special trailer was constructed to limit th height of the load for passing under bridges, the overall length of th lorry and trailer being 70 feet. On arrival at the site, the lorry wa backed under temporary girders which formed an extension of th overhead bridge. A pair of gantry cranes running on the top of th bridge lifted the gate, traversed it to its appropriate bay, and lowere it into place. The method was very convenient and saved a grea deal of time.

The next point of interest was the provision in the six main dam of the 60-inch diameter outlet-valves of the mechanically-operate needle type, in which energy-dispersers were incorporated. Thos dispersers were of the free-vortex type, and dissipated the energy the escaping jets so as to protect the downstream bed from erosion A typical example was shown in Figs. 5, Plate 1. When dischargin under full head, the hydraulic energy destroyed amounted to approx mately 4,000 h.p. There were altogether eighteen free-vorte dispersers in the Galloway scheme for the protection of free-discharg outlets. By placing the regulating valve at the downstream end the steel-lined pressure-culvert, the velocity was limited to a sa figure which would ensure the minimum of erosion, and cavitatio was completely eliminated. At the upstream end of each culvert bellmouth inlet-casting was provided with a prepared face to recei an emergency closing gate to isolate the culvert and regulating val for inspection or repair. The gate (*Figs. 47*) was of the most rece

ype, consisting entirely of tubular steel rollers spanning the width Mr. Ball. f the bellmouth opening, rotating on gunmetal bushes and carried y axles held between side frames. The horizontal spaces between ae rollers were stanched by bronze rollers of smaller diameter, eld tightly in contact with the main rollers by the water-pressure, nd being rotated by them. The spaces between the ends of the ollers and the side frames, which were faced with gunmetal, were mited to about 0·015 inch. It was only through those spaces, which ormed the working clearance, that any leakage could occur, and such as the effectiveness of the design that the tightness was almost

Figs. 47.



FREE-ROLLER GATE FOR OUTLET-CULVERT.

erfect. When the gate was closed under full head a fine mist only as discharged into the tunnel. By virtue of its free-rolling antiiction design, the gate closed by its own weight under the full nbalanced head, and was therefore in a true sense an emergency ate. The gate was normally housed above top water-level, and ould be lowered under the action of gravity by releasing a clutch a the headstock, the speed of closing being limited by a centrifugal rake to about 15 feet per minute, so that an outlet could be closed a an emergency in 5 minutes after releasing the clutch.

Dr. W. L. LOWE-BROWN said that the late Colonel McLellan's Dr. Lowe-Brown. reat share in the conception and design of the Galloway power cheme had been well expressed by Lord Meston. The wording of

Dr. Lowe-Brown.

the memorial plaque at the Tongland station was also eminen
appropriate—

1874 1934

WILLIAM McLELLAN
C·B·E. M·I·E·E.
OF ORCHARD KNOWES
KIRKCUDBRIGHTSHIRE
TO WHOM THE
CONCEPTION OF THIS WATER
POWER SCHEME WAS DUE
A MASTER OF ENGINEERING
HE DEVOTED HIS TALENT
TO THE DEVELOPMENT OF
ELECTRIC POWER IN MANY
PARTS OF THE WORLD

*"Si monvmentvm reqviris,
circvmspice."*

Mr. Williamson.

Mr. James Williamson remarked that it had been his privil
to be connected with the scheme under Sir Alexander Gibb
Partners from its very inception in 1923. Lord Meston
spoken very kindly of the engineers and their work; he would
to add that the engineers had been fortunate in having a boar
directors to work with who appreciated the problems involved
were willing to allow for difficult circumstances.

The first suggestion of the Galloway scheme had been on rath
small scale; it was proposed to develop power at loch Stroan on
Blackwater of Dee. That original suggestion led to the whole a
being examined. There was then no immediate prospect of any ou
for the power, but intermittent studies were made of various meth
of developing the area, and it was seen that if loch Doon could
utilized in addition to Clatteringshaws for storage a very effect
and comprehensive scheme could be drawn up. Loch Doon, howe
was at that time reserved for the river Doon hydro-electric sche
proposed by the Burgh of Ayr. Later, the introduction of
Grid scheme appeared likely to provide an outlet for the power,
it was found that the river Doon scheme was unlikely to be car
out. In 1927, therefore, a memorandum was prepared coverin
comprehensive scheme including storage in loch Doon. T
memorandum compared two methods of development, one
continuous load and the other for 50 per cent. load-factor, and furt

udies based on that comparison led to the consideration of the area Mr. Williamson.
r peak-load generation. The extraordinary fact was found that
n the whole development as then contemplated continuous load
ould cost about £65 per installed kilowatt but that for 50 per cent.
ad-factor the cost for the additional capacity would be £9 per
lowatt, and it appeared that any further expansion of capacity for
wer load-factors could be obtained at less than £9 per kilowatt.
ull consideration of all the circumstances led to a load-factor of
) per cent. being adopted, which corresponded to an installation of
bout 100,000 kilowatts. One of the deciding reasons was that
)0,000 kilowatts was the capacity of two single lines of the Grid;
nother reason was that when operating on 20 per cent. load-factor
hort high-peak flows would be discharged down the river every day
the winter, and higher discharges would have involved the risk of
amage to river-banks and riverside property. That aspect of the
atter also limited the installed capacity.

*** Considering next some of the matters dealt with by Messrs.
udson and Hunter, Mr. Williamson observed that the estimates
r run-off constituted one of the most important sections of the
timates for a power-scheme. The mean rainfall-figure of 64 inches
r the whole catchment-area (Table I, p. 329) agreed well with the
gures estimated in the early stages of the scheme from the Clatter-
gshaws and Knockingarroch gauges[1] and confirmed roughly as
e result of a few months' records of a number of new gauges in 1929.
he Authors' method of arriving at mean rainfalls on the Dee catch-
ent-area above Glenlochar (Table II, p. 330) was not very good when
pplied to ascertaining differences. There was purposely a pre-
onderance of gauges on the high-rainfall areas which was not
alanced by similar close distribution on the low-rainfall areas, there
eing in fact an area of over 100 square miles on the eastern side of
e catchment without any gauges. It would be better to average
e rainfalls of the gauges by taking account of the area for which
ch gauge was representative. By that method the following
gures had been arrived at :—

	1930.	1931.	1932.	1933.	1934.	1935.	Average.
ean rainfall : inches	72·3	68·5	69·8	49·4	71·0	65·2	66·0
un-off : inches .	57·9	58·9	58·1	37·9	58·2	51·2	53·7
otal loss : inches .	14·4	9·6	11·7	11·5	12·8	14·0	12·3

*** The following part of this contribution, and the succeeding
ontributions, were submitted in writing.—Sec. Inst. C.E.

[1] James Williamson, *Correspondence on* "The Areas Covered by Intense and
idespread Falls of Rain." Minutes of Proceedings Inst. C.E., vol. 229
929–30, Part I), pp. 187–193.

Mr. Williamson.

The losses thus found were more consistent from year to year t the results often found on other catchment-areas, and the average brought out was 12·3 inches. In preparing the estimates for po it had been assumed that the average loss would be about 12 inch it was realized from the geological conditions that very little wa would be lost underground.

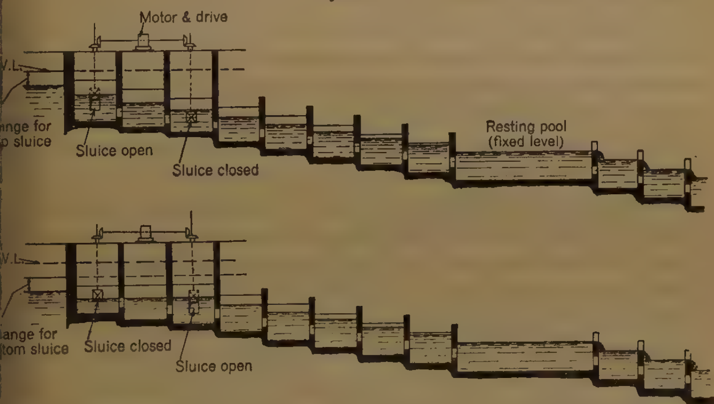
A study of some of the longer-period gauge-records in Gallow raised the question of whether there was indication since the beginn of the present century of a change of climate, resulting in milder wetter winters and increase of rainfall. The gauge at loch Fin had been read since 1885. In the first 18 years of records, till 19 the average rainfall recorded was about 49 inches and only records were above the long-period figure of 55·1 inches given *Fig. 2* (p. 328). It was generally assumed that any period of years covered a sufficient number of high and low variations to prod a long-period record approximately the same as any other eq period. In the 35 years from 1903 to 1937, however, there w only about 6 years with records below 55 inches, and the aver was about 59 inches. That was very much more than the 49 inc of the previous 18-year period, and substantially more than figure of 55·1 inches adopted as the standard. Over the 35-y period it represented an excess of 2½ years' rainfall. The point of considerable general interest, apart from its particular interes the water-power authorities, and he suggested that it was worth the attention of the Meteorological Department. If the conclus were reached that the dictum " The weather always pays back w it owes " still held good, then it would have to be recognized tha very large overdraft had been accumulated, the effects of wh would be felt when it was called in.

He desired also to call the attention of the meteorological authori to the rainfall-map, *Fig. 2* (p. 328), which had evidently b prepared with close attention to physiography and exposure. rainfall at Forest Lodge was there given as 63·2 inches. Over period of the Galloway Company's records, which for that gauge been obtained from the Meteorological Department, the indicati were that the rainfall at Forest Lodge was rather higher than Clatteringshaws, and that the long-period value should be ab 71 inches instead of 63·2. He recollected having been informed t at one stage doubt arose as to whether the gauge at Forest Lo was recording the full rainfall, that the location was investiga and a shift arranged, and that the subsequent readings had been a higher scale. If adjustment of the 70-inch contour were to made to allow for that change, it would be moved 2 miles eastwa at the words " Forest Lodge " on the map, and a suitable adjustm

uld also need to be made on the 80-inch and 90-inch contours. The location suggested would be consistent with the fact that Forest dge was in the lee of the highest part of the Kells range of hills. Mr. Williamson.

With regard to the provision made for the passage of fish, the thors expressed the view on p. 352 that, in general, passages of the er-fall weir type were more easily regulated and cheaper to con- uct than passes of the submerged-orifice type. Their conclusion s open to doubt, and did not appear to be applicable to the ladders Tongland, Earlstoun, and Carsfad. A special feature at Tongland s the large daily rise and fall of the reservoir, amounting sometimes 10 feet within a few hours, and the necessity for maintaining equate passage-way to the reservoir and a nearly constant flow all stages. No reasonably effective and economical method of

Figs. 48.



PRINCIPLE OF REGULATION OF FISH-LADDER WITH SUBMERGED ORIFICES.

ing that with the over-fall type of ladder had been found, but omerged-orifice discharge appeared to afford the means of ensuring e necessary regulation, making the passage for fish easier, and owing a shorter ladder to be used. The principle of the regulation s shown diagrammatically in *Figs. 48*. Below the top resting ol the ladder had a regular form of construction in steps of 2 feet height, with chambers 15 feet long and 6 feet deep and a sub- rged orifice of constant size in each cross wall. The minimum antity of water sent down should be sufficient to fill the pools and ow a small surplus to pass over the top of the cross walls. For y slight fluctuation of quantity above the minimum the surplus uld also pass over the cross walls, and the chambers would always full, with a 2-foot drop in level from chamber to chamber. The gulation of the quantity and the provision of a suitable opening

Mr. Williamson.

to the reservoir at all stages of water-level was done by meani the top flight of chambers above the uppermost resting pool. . those chambers the cross walls were carried higher than in the nor ladder and any possibility of spilling-over was eliminated. orifices in the walls were all of the same size and shape, so that t had the same coefficient of discharge. If then a sluice were ope to admit water at the top of the ladder, the flow would adjust i to steady conditions, and would produce equal fall from chambe chamber down to the top resting pool, in which the water-level l practically constant. The fall would vary slightly during operat as it depended on the reservoir-level and the particular access-sl which was in use. The upper diagram of *Figs. 48* showed t maximum and minimum levels in the chambers for an assumed (in reservoir-level of 3 feet with the water passing through the up of the two access-sluices, and the lower diagram showed the co sponding levels for a further drop in reservoir-level of 3 feet v the water passing through the lower access-sluice. The change f one sluice to the other was effected automatically by a float- trolled motor, the closing of one and the opening of the other ceeding simultaneously without interruption of flow.

Figs. 48 were diagrammatic only, for the purpose of illustra the principle. At Tongland there were actually three slu openings for the range of 10 feet, only one of which was open a time. At Earlstoun and Carsfad there were two sluice-openings a maximum range of 8 feet, but normally only the upper open was in use, as the fluctuation of level was small, except in emergen The proposed arrangements had been considered satisfactory by fishery experts and a 2-foot drop from chamber to chamber had b agreed to. In discussions with the experts on the over-fall typ ladder it had been generally indicated that they would like cham 20 feet long furnished with internal baffle-walls and with a fall f chamber to chamber not exceeding 1 foot 6 inches. If those quirements had prevailed at Tongland they would have entailed increase of 400 feet in the length of the ladder, which would have ad very largely to the difficulty and the cost. (A small ladder of overfall type with long pools and baffle-walls was shown leading from the tail-race of Tongland power-station in Figs. 18, Plate There was never any doubt that fish would pass easily thro the orifices under a 2-foot head, as instances were kn where salmon passed through sluice-openings under much gre heads. The helical ladder at loch Doon had been devised meet exceptional circumstances at a time when the work the dam was in active progress. The major difference betw the operation of the reservoirs at loch Doon and at Tongl

Mr. Williamson.

s that the rise and fall at the former was slow and seasonal t of large range up to 40 feet, whereas at the latter there s a rapid daily rise and fall of up to 10 feet which called for tomatic sluice-control. The slower movement at loch Doon rmitted of manual adjustment of the sluices at intervals of a day two. The helical-tower type of ladder would appear to be capable adaptation to almost any range of rise and fall in a slowly-ctuating reservoir.

The large use that had been made of the horizontal-arch type of m was noteworthy. That type was particularly useful for suit-le sites, and it had decided advantages where foundations might ve to be taken deeper than expected, the additional work then volved being far less than that required in deepening a gravity m. In most of the Galloway dams the arch abutted against a ep face of rock on one side and against a gravity-dam on the her. It might normally be preferred to abut against rock on both des, but the large spillway-capacities required led in certain cases the deliberate selection of sites on which a long length of low avity spillway-dam could be developed on one side. The sectional nstruction and arrangement of joints and expansion-joints received pecially careful consideration.[1]

Tunnelling formed the key to two essential parts of the Galloway heme. The development in tunnel technique had been rapid in cent years, and with increase in the speed of driving and lining ere had been decrease in the cost. For lined water tunnels be-veen 8 and 12 feet in diameter, the cost per foot 12 years ago ould have been about £1⅓ per foot of diameter, but in Galloway e cost per foot had been less than £1 per foot of diameter. A otable achievement by the contractors for the loch Doon tunnel as the simultaneous driving and lining at a rapid rate of a tunnel nly 8 feet 3 inches in diameter, executed from one end only.

The introduction at Earlstoun and Kendoon of reservoir canals pable of dealing with the daily fluctuation of the reservoir and roviding full-load capacity even at the lowest working level, while ot accompanied by material saving of cost, produced advantages other respects and led to increase of the power-output. At endoon the average working head had been increased from the nticipated figure of 150 feet to 157 feet, producing an increase of ower of nearly 5 per cent. together with an increase of capacity. ther advantages were the greater facility of control of the station, e more convenient location of the intake, the easing of the surge-

[1] J. Williamson, "Design and Waterproofing of Shrinkage, Contraction, and xpansion Joints in Concrete Dams." World Power Conference, Washington, 936.

Mr. Williamson.

tower problem, and, at Earlstoun, the elimination of additio gate-control (the free-roller gates at the forebay at the lower of the canal constituting the turbine-valves). Where turbi valves were placed at some distance away from the machines, whole of the water-hammer pressure produced by the rapid shutti down of a machine had to be taken by the pipe. That conditi arose at Tongland, Earlstoun and Carsfad, where the pipes had b designed for pressures of the order of 60 to 70 per cent. above maximum static pressure. Where relief-valves were installed, as Glenlee and Kendoon, the excess pressure to be provided for much less.

Referring next to the Paper by Messrs. Hawthorne and Willia Mr. Williamson observed that it did not appear to give full wei to the large part that could be taken by the seasonal storage the scheme of regulation—the Galloway development was only i very minor sense a run-of-river scheme. The storage problem in no sense comparable to that of a water-supply scheme or even a hydro-electric scheme for continuous load. In those cases wa was stored to give a continuous and even supply of water, and pipe systems themselves in conjunction with the capacity provid put a limit on the quantity that could be drawn. In the Gallow scheme, with a series of stations having widely-varying heads a with seasonal reservoirs at different elevations, the problem was to provide a regulated supply of water as such but to provide regulated supply of power; an additional factor was thus introdu by the varying head. Still another factor was the seasonal use power, large in winter and small in summer, which admirably suit the average run-off characteristics of the area. That co-ordinati of power-output to run-off effected a very large saving in the quant of storage required. The seasonal storage was provided on the ar of highest ground with the highest rainfall extending along t western boundary of the catchment; the total storage-area was abo 125 square miles, but 20 square miles of the loch Doon area had be allocated to providing water for Ayr and the river Doon, so th the net area for seasonal storage was 105 square miles, about 27 p cent. of the total catchment-area. In an average year the pow provided directly from storage amounted to about 125 million un —over 60 per cent. of the total power—from 27 per cent. of the ar The quantity of stored water that had to be spilled from t seasonal reservoirs was negligible, and no run-of-river water w dealt with in the Glenlee station. The quantity of run-of-riv water to be dealt with in the other four stations increased fr Kendoon downwards, and more than half of the whole quanti passed only through Tongland station at the small head of 105 fe

Mr. Williamson.

he quantity of power which could be generated from the run-of-ver water was estimated to amount in an average year to about) million units, allowing for inevitable flood-spilling, and was onsiderably less than that from the stored water, although the total uantity of water required to produce it was much greater. A large art of the run-of-river water, estimated at 60 per cent., could, owever, be regulated by proper manipulation of the storage by the llowing method. So long as the natural flow in the river did not xceed 20 to 25 per cent. of the power-station capacity, it was only ecessary to supplement the flow by discharge from the storage-eservoirs and bring it up to the requisite capacity. In dry spells in inter the discharge had to be heavy, and in wet weather small or nil, ubject to the requirement that during the heaviest load-period for short time in winter Glenlee had to be operated to provide power, ven if there was sufficient natural water to run all the other stations. he result was that of the total power-output in a year producing)5 million units, consisting of 125 millions from storage and 80 illions from run-of-river, about 170 millions should be capable of most complete regulation for peak load.

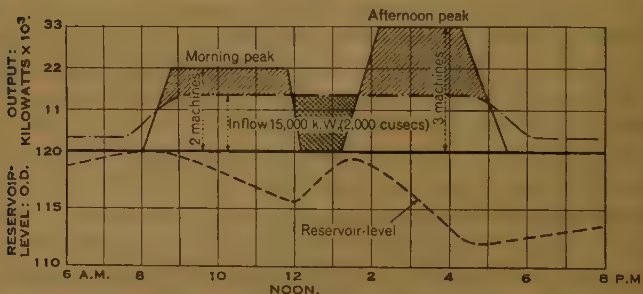
The fact that the stations when run continuously could consume our to five times the average river-flow and the resulting output ould generally be absorbed ensured a very high utilization of the vailable water. In the first year's operation it appeared that loss uring floods amounted to roughly 10 per cent. of the run-of-river ow. When the estimates for the scheme had been prepared it ad been assumed that there would be times, particularly at week-nds, when it would not be convenient for the grid to take all the ower capable of being produced; the utilization-factor had there-ore been taken at 80 per cent. for the run-of-river water, whereas) per cent. had now been found possible, the increase of output eing about 10 million units. Conservation in the estimates and arious minor modifications effected on the original scheme had dded a further 8 million units. The product of the area in actual ater appeared to have been very closely estimated. By careful ogging of all details of operation of the scheme it should be possible o maintain a useful comparison between aggregate rainfall and ggregate run-off on the whole catchment-area.

The control of the daily reservoir at Tongland was interesting. he Authors stated that the capacity of the reservoir was 2 hours' ull-load running of the station, and that with a river-flow corre-ponding to half load and starting with a full reservoir the station ould run on full load for 3 hours. Should not the answer to that alculation be 4 hours? In any case, statement of the capacity in hat manner did not conduce to a realization of the capabilities of

Mr. Williamson.

the station for taking its share in the load-system. In the pr liminary studies it had been assumed that it would suit the gene requirements in winter if the station were operated at two-thir load on the morning peak and at full load on the higher afterno peak. *Fig. 49* showed the nature of the daily load assumed, wi the station shut down at the dinner-hour recess when the industr load fell to a low figure. It would be seen that that system operation could be maintained without exhausting the storag The inflow shown in *Fig. 49* corresponded to 15,000 kilowatts, whi was somewhat less than half load. The capacity of the station f peak load might be stated as equivalent to 6 hours full-load runni on a 9½-hour base. He understood that in normal conditions winter the station was run on a method similar to that describe

Fig. 49.



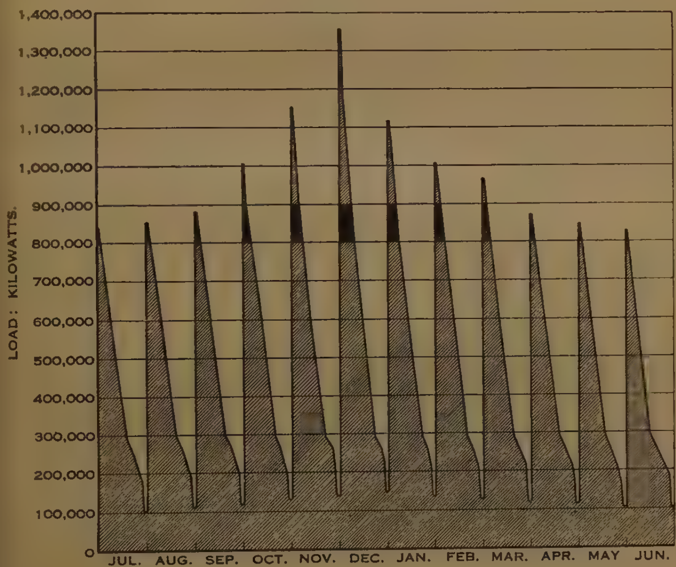
Tongland Power-Station: Operation of Daily Storage.

On pp. 379–380 the Authors mentioned three alternative metho of providing power for peak loads. It was not too much to say th any hydro-electric scheme which could be contemplated for co tinuous load would be much improved if enlarged and operated f peak load, provided the necessary demand existed. The Gallowa scheme would not have been economical on continuous load b became competitive with steam when arranged for peak load. A an illustration of the change in cost per unit of capacity which coul take place, Glenlee station, if arranged for 6,000 kilowatts of co tinuous load, would have cost about £75 per kilowatt. The actu installation was 25,000 kilowatts, the extra 19,000 kilowatts of capacit having been obtained at a cost of about £10 per kilowatt; furth increase in capacity, which could usefully have been employed b which it was now too late to contemplate, could have been obtain for less than £10 per kilowatt. By means of pumped storage plan it appeared to be possible at suitable sites to obtain capacity f overtaking the finer peaks of a system at a cost per kilowatt w below the lowest cost which appeared possible at present for co

parable steam operation. *Fig. 50* had been prepared to indicate a rough approximation to the combined load-system of Central Scotland and North-West England at a recent period; for the sake of simplicity, each month's operation had been concentrated into a single peak with a height corresponding to maximum load at the middle of the month. The diagram showed the large increase in load in winter as compared with summer, the whole of the increase being of the fine peak-load type which was the most expensive to

Mr. Williamson.

Fig. 50.



SEASONAL VARIATION OF PEAK LOAD.
(Each peak represents monthly aggregate load.)

produce in steam stations. The black areas indicated the load that could be taken by the Galloway scheme to replace 100,000 kilowatts of steam plant. It was evident that a large amount of expensive peak load still remained.

With reference to Mr. Mountain's Paper, Mr. Williamson observed that the remarkable feature brought out was the great distance over which the output was transmitted, the bulk of it being fed to two load-centres 240 miles apart, over a single 132-kilovolt line. He understood that the use of a long single line to transmit 100,000 kilowatts was only rendered possible because the Galloway stations were situated somewhere near the middle of the line. Was it the case that if the Galloway stations had not existed the capacity of the line

Mr. Williamson.

for interchanging power between Scotland and North-West Engla[nd] would only have been 20,000 or 25,000 kilowatts, so that from o[ne] aspect there had been effected a four or five-fold increase of capacit[y].

Figures given on p. 419 suggested that the cost of modern stea[m] generation might be taken as £2·5 per kilowatt plus 0·085*d*. per uni[t]; the latter figure was probably conservative, having regard to t[he] present price of coal. Assessing the annual value of the Gallow[ay] stations on that basis :—

102,000 kilowatts at £2·5 per kilowatt .	£255,000
200 × 10^6 units at 0·085*d*. per unit .	£71,000
Total annual cost of steam generation .	£362,000

The annual cost of the Galloway scheme had been stated as und[er] £240,000, so that there was a considerable margin in its favour.

The Author suggested an investigation of hydro-electric schem[es] for low load-factors. Unfortunately, each hydro-electric site co[n]stituted a separate problem, and no general comparison could [be] made. Some of the essential elements that conduced to sma[ll] capacity-cost on low-load factors were the following :—

(i) Increase of capacity involved no increase in prelimina[ry] and promotion expenses.

(ii) Increase of capacity required no extra expenditure for lan[d].

(iii) Extra capacity in pipes was obtained at a lower rate tha[n] for continuous load.

(iv) Extra capacity in tunnels was obtained at a much low[er] rate than for continuous load.

(v) There was generally a slight saving in cost per kilowatt [of] plant and buildings for power-stations with increase [of] capacity.

(vi) If capacity were required mainly for winter peak load, the[re] might be a saving in the cost of storage as compared wit[h] that required for continuous load, in situations such a[s] the west of Scotland.

For heads of 300 to 600 feet, which were very convenient f[or] development, the extra cost of power-station and plant would [be] of the order of £5 to £4 per kilowatt, provided the units were n[ot] small. The extra capacity in pipe-lines and tunnels of modera[te] length might be obtained for £2 to £4 per kilowatt. The range [of] total extra cost on those figures was therefore from £6 to £9 p[er] kilowatt, but might be more if the supply-conduit were long.

Mr. Bridge.

Mr. N. C. Bridge observed that Mr. Mountain contended that t[he] cost of power from the Galloway stations, operating at 20 per cen[t]. load-factor, was only about 75 per cent. of the cost of the sam[e] power from a modern steam station. That was surely very muc[h]

open to question, and did not seem very sound ground upon which to base the advantages of a hydro-electric scheme. The figures given on p. 419 for a steam station could be shown to represent an operating charge (for fuel, etc.) of 0·0875*d*. per unit supplied, plus an annual charge for interest and depreciation of £2·46 per annum per kilowatt of plant installed. The cost figure given for the hydro-electric scheme represented £2·35 per annum per kilowatt, of which much the greater part, possibly £2·25, would represent capital charges. The capital charges for the hydro-electric scheme would thus be less than those for the steam station, which seemed absurd. There could be no difference in the interest-rate, and though the overall depreciation-rate for the steam station might be double that for the hydro-electric scheme, the capital outlay to which the rates were applied would not, for the steam station, be more than about half that for the hydro-electric scheme—otherwise it could only be considered to be a very extravagant station and hardly of the type that might have been put down in lieu of the Galloway scheme. If £1·5 per annum per kilowatt were taken as a more reasonable estimate of the capital charges for the steam station, and the rather higher figure of 0·1*d*. per unit as the running charge, the total cost per unit with 20 per cent. load-factor became approximately 0·305*d*., as against Mr. Mountain's suggested figure of 0·425*d*. and the Galloway scheme figure of 0·32*d*. With increasing load-factor, the hydro-electric scheme costs would, of course, show up to greater advantage. Mr. Bridge.

Such comparisons as the foregoing were, however, only part of the story. Taking a broader view, there would seem to be a sound case for developing water-power on a combined steam and hydro-electric scheme where (i) the assured load-factor on the hydro-electric plant would bear a satisfactory relationship to the capital outlay; (ii) the hydro-electric plant could be operated to level out the load-curve on the steam stations and so improve their operating efficiency; (iii) the availability of the hydro-electric plant (having regard to floods, droughts, frosts, etc.) would bear a satisfactory relationship to the steam-plant reserve; and (4) the costs and losses of transmission to the centres of load-demand would not outweigh the gains on other counts.

Mr. Mountain's Paper provided a welcome assurance upon the technical questions arising out of the transmission and interconnexion problems of the Galloway scheme, but would seem to lack conviction upon some of its economic aspects. That conviction might perhaps come in time, and the foregoing comments should not be taken as intending any disparagement of what was undoubtedly a highly creditable enterprise.

Mr. W. A. Coates observed that Mr. Mountain had given a figure of 0·32*d*. as the average cost per unit at the generating station Mr. Coates.

Mr. Coates. busbars. A closer comparison would have been provided by con puting the cost at the point of usage, which was where any alternati peak-load steam station would have been located. From the da for additional capital and transmission-losses given in the Pape it would appear that the cost per unit at the receiving station bars would be approximately 0·38*d*. That figure was still on the rig side as compared with a steam station having the same load-facto it would, however, be unwise to draw any general conclusions fro the fact, because the capital cost of the Galloway development wa exceptionally low. Conditions of rainfall and the service to whi the plant was to be put made possible the virtual elimination water-storage. For a hydro-electric plant which was expected operate on normal daily loads, a mere 12½ days' supply of wat behind the dams could not be risked. Adequate water-stora for such conditions would no doubt have brought the capital co of the scheme to about the usual average of £40 to £50 per kilowat

It was difficult to get a clear idea of the real effect of the develo ment on the finance of electricity-supply in Great Britain witho knowing whether there were in fact plants available in the Glasgo Manchester, or Liverpool areas on which the capital was largely wholly amortized so that the generating costs would be exceptional low. A somewhat disturbing possibility from the point of view cost was hinted at on p. 395 by Messrs. Hawthorne and Willia when they mentioned the Central Electricity Board taking supp at short notice to avoid spilling to waste. Were base-load stea stations operated below their most efficient load-factor on th account? If so, the overall cost per unit might be increased consequence. The point that a hydro-electric station could be p on and taken off load quickly was well taken. Probably a diese engine station alone could give similar service, and the fact that fuel had to be imported was a strong point against that class station. That consideration supported Mr. Mountain's suggesti of an extended hydrographic survey.

The statement by Messrs. Hawthorne and Williams (p. 393) th the Galloway plants were of value in regulating frequency in Centr Scotland and North-West England was surprising. Dalmarnoc Clarence Dock, and Barton power-stations were all working wi controlled frequency, and each was comparable in size with t entire Galloway development. As the governing on steam turbines w usually very much closer than that possible with water turbines, sure the network frequency would be determined by the steam station

Mr. Halcrow. Mr. W. T. HALCROW observed that the Galloway hydro-elect plant, which supplied power for general purposes, differed from th carried out in the Highlands of Scotland in that it was designed

operate at a low load-factor of about 20 per cent. The Grampian plant also supplied current for general purposes, but operated at a load-factor of about 50 per cent. The Kinlochleven and Lochaber plants provided current for the production of aluminium, and worked on a load-factor approaching 100 per cent. The Galloway plant served a useful function as a feeder of the "grid," and would be unsuitable for electro-chemical industries. The low load-factor arose from the fact that the topographical conditions did not permit of the formation of storage-reservoirs of sufficient capacity to regulate fully the run-off from the catchment-areas, so that the cost of the work as seen from the figures given in the Papers was high. As practically the whole cost of current from a water-power station was interest on capital and depreciation, the cost per unit was known when the works were completed, and was not liable to an upward fluctuation as in the case of a steam or oil plant where the fuel-cost might rise. Also, by the operation of a sinking fund, the cost would steadily be reduced over a period of years. That was an important advantage of water over steam power. Mr. Halcrow

In the Highland schemes power could be developed on a continuous-load basis, as it was possible to construct reservoirs of large capacity at low cost. Such power could be produced at $\frac{1}{8}d.$ per unit and less, and in consequence chemical or electro-metallurgical industries, such as the production of aluminium, could be established. Efforts were being made to establish a calcium-carbide industry, but unfortunately landowners and others had so far successfully blocked the Bill promoted in Parliament for the necessary statutory powers to construct the hydraulic works. Many engineers and others interested in Bills of a technical character doubted the wisdom of Parliamentary procedure which allowed such Bills to be rejected on second reading before being examined by a Select Committee that would hear the evidence of experts for and against the Bill. It had been suggested in some quarters that Highland water should be developed generally for power-supply purposes. If specialized industries such as aluminium and carbide production could only be established in Great Britain if cheap power from water were available, there would appear to be some argument in favour of reserving the power for such purposes. The difference in cost of production of power between $\frac{1}{8}d.$ and $\frac{1}{4}d.$ a unit might make or mar an electro-chemical scheme, but would be of small importance where the energy was distributed for general use and where rates of $6d.$ and upwards might be charged.

The works described in the Papers contained many features of interest. The adoption of the combined arch and gravity dams to a large extent was noteworthy; had the designers taken into con-

Mr. Halcrow. sideration in their calculations the cantilever action of that part of the arch which was held by the rock foundation and was consequent not free ? Also could the Authors give reasons for the alignment of the Carsfad dam, shown in Fig. 8, Plate 1, and *Fig. 9* ? It appeare that a straight dam at that site would have been more economica

On p. 338 there was described the lining of a canal with concre placed in situ on slopes of 1½ to 1. The concrete had apparent proved not to be watertight and had required subsequent treatmen On the Grampian hydro-electric works where a large canal was line in a like manner some leakage was measured, and he had little doub that the trouble arose through the difficulty experienced in makin homogeneous the concrete laid on the 1½-to-1 slopes. It was no possible to ram it, and reliance had to be placed on the skin forme by screeding to make it impervious. In that case the difficult had been satisfactorily overcome by coating the surface with special cement, and there had been no trouble since the aqueduc had been put into use about 4 years ago.

The description of the fish-passes, which were, he believed, th largest in Great Britain, should dispel the fears of those who mai tained that if the natural conditions in a river were interfered wit fishing was bound to be adversely affected. His experience was tha there was no difficulty in providing channels for fish to pass a dam but that there was controversy between engineers and fishin experts as to the best form of pass and the minimum quantity o water required to operate it. He hoped that that divergence o opinion would be narrowed as a result of the work now being unde taken by the Institution Research Committee on Fish-Passes.

On p. 361 the Authors referred to the use of diesel locomotives i the Glenlee tunnel. Had any trouble been experienced due to th poisonous gases which such machines discharged ? Was the force ventilation at the working faces required because of the locomotives His experience was that, where electric locomotives were used, th discharge of air from the drills at the face was sufficient to keep th tunnels fresh.

In the early stages of construction of the dams rapid-hardenin cement appeared to have been used, and trouble arose from excessiv shrinkage and cracking of the concrete. He would consider rapid hardening or quick-setting cement of any kind unsuitable for dam or similar large concrete structures, and it was interesting to lear that those cements had been abandoned and that cement groun more coarsely than that normally supplied by manufacturers ha been used, as for the Laggan dam. It appeared from the remarks o p. 366 that that change had proved effective in reducing the develo ment of shrinkage-cracks. It was hoped that the research now bein

carried out on low-heat cements would result in the production of a cement that would have a minimum of shrinkage on setting. Mr. Halcrow.

Mr. DAVID LLOYD observed that the values of rainfall, run-off and loss given for the Galloway catchment would afford a comparison with the data provided in two recent Papers. Accurate observations for the Severn catchment [1] indicated that the average total losses were about 18 inches from a long-average rainfall (1881–1915) of 4·6 inches; on the Vyrnwy catchment (Montgomeryshire) it had been found [2] that the total losses were about 19 inches from the long-average rainfall (1881–1915) of 70 inches. The Authors had observed that the total losses on the Galloway Dee were of the order of 13 inches with a long-average rainfall of about 63 inches. There would, at first sight, appear to be no basis for reconciling the different values, but he would suggest that they could be largely explained by differences in the climate (temperature, sunshine, etc.) experienced at each catchment and also the geological character of the underlying strata. Regarding the weather, the conditions were rather more severe over the higher catchment in Scotland than in the Midlands of England. The underlying strata at Galloway were less porous than those underlying the Severn basin. A study of the total losses at various areas led him to suggest that total loss should be considered as being made up of a primary loss or "pellicular" loss from a zone at the surface, and a deferred or ground-water loss. It might be that, whilst the pellicular loss at the Galloway catchment and that at the Severn catchment were comparable as a function of the evaporation-opportunity and the weather, the ground-water loss at the Galloway catchment overlying impermeable strata was very low. That reasoning might afford a partial explanation of the low value of losses experienced over the Galloway Dee, but it might not explain it wholly, as he had noticed when applying differences expected because of different weather and geophysical character. Mr. Lloyd.

The method used by the Authors to compute the general rainfall over the area was that described as a provisional method by a committee on the determination of rainfall.[3] That provisional method was based on the observed fact that, as a generalized truth, the percentage rainfall over an area in a given period was sensibly constant. On a large area, Dr. John Glasspoole had observed discrepancies,[4] and

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[1] S. M. Dixon, G. Fitzgibbon and M. A. Hogan, "The Flow of the River Severn," Journal Inst. C.E., vol. 6 (1936–37), p. 81 (June 1937).

[2] D. Lloyd, "Rainfall and Loss over Vyrnwy Catchment Area," Quar. Jour. Roy. Met. Soc., vol. 62 (1936), p. 219.

[3] "Report of the Joint Committee to consider Methods of Determining the General Rainfall over Any Area." Trans. Inst. Water Engineers, vol. xlii (1937), p. 231.

[4] J. Glasspoole, "The Rainfall of Norfolk." British Rainfall, 1928, p. 278.

Mr. Lloyd. also, from considerable study and use of the methods, it appea[illegible] to Mr. Lloyd that the provisional method was reasonably accur[illegible] only on a moderate-sized area. On areas of 200 square miles or m[illegible] with few rain-gauges, he had noticed that departures of values gi[illegible] by the provisional method from those given by the cartographi[illegible] method were not within reasonable limits, particularly when the l[illegible] (a residual) was extracted. The annual general-rainfall values gi[illegible] in Table II (p. 330) might thus be 1 or 2 inches low, and subsequen[illegible] also the values for total loss; hence it would be of value if [illegible] Authors would indicate the limits within which the computed value l[illegible]

Mr. Hunter. Mr. Hunter, in reply, observed that Mr. Binnie had asked w[illegible] method had been adopted in computing the discharge of the o[illegible] standing flood that had occurred at Earlstoun on the 1[illegible] December, 1936. There had, in that instance, been no spe[illegible] difficulty in arriving at a reasonably accurate estimate, since [illegible] whole of the flood had been discharged over the spillway-crest of [illegible] dam and through the power-plant and floodgates. The considera[illegible] reduction that had been effected in the magnitude of the fl[illegible] lower down the river by the regulating effect of loch Ken certai[illegible] appeared remarkable at first sight, and was due, partly to the f[illegible] that the loch was at the time standing at a low level, and par[illegible] to the short duration of the flood in question. Sustained floods w[illegible] not attenuated to the same degree.

Both Mr. Williamson and Mr. David Lloyd had criticized t[illegible] method adopted by the Authors in computing the mean ann[illegible] rainfall over the catchment-area. On plotting on a map the ann[illegible] percentage-values of the different rain-gauges it had been impossi[illegible] to trace any consistency or to find any basis on which percenta[illegible] contours could be drawn with any confidence. In order, therefo[illegible] to avoid the labour of preparing separate isohyetal maps for ea[illegible] year, the simple method described had been adopted, and in vi[illegible] of the distribution of the gauges it was believed that the error wo[illegible] not be serious. The alternative method proposed by Mr. Williams[illegible] in which a weighting factor was applied to the individual percenta[illegible] values, had been considered, but the Authors had felt doubtful [illegible] the circumstances whether the results were likely to be any m[illegible] reliable, and it was to be noted that the figures for the annual los[illegible] as calculated by Mr. Williamson when plotted to a base of ann[illegible] rainfall were not less irregular than those given by the Authors.

They were interested to note Mr. Williamson's reference to [illegible] rainfall at the Forest Lodge gauge, since they had also come to [illegible] conclusion that some change in the location or exposure of [illegible] gauge had occurred in the past, and that the true average value [illegible] appreciably higher than that stated by the Meteorological Off[illegible]

hey had felt, however, that as the matter was somewhat conjectural nd as the probable error would have but a negligible effect upon the omputation of the mean annual rainfall for the whole catchment, revision for the purposes of the Paper was hardly justifiable. Mr. Hunter.

The Authors' suggestion that fish-passes of the overfall-weir type ere more easy to regulate than those incorporating submerged rifices referred only to the main portion of the pass, which had to eal with steady conditions. Where a fluctuating reservoir-level ad to be dealt with, as at the head of the passes at Tongland, arlstoun and Carsfad, the submerged orifice was, as Mr. Williamson ointed out, the only satisfactory method. For the lower portion f the pass, however, the overfall-weir type of pool possessed the dvantage that the depth of water could be maintained practically onstant without the use of sluices between the chambers should it e desired to operate it at a considerably reduced flow.

The Authors wished to thank Mr. Williamson for the most nteresting contribution that he had made to the Discussion, which dded materially to the value of the Paper.

With regard to Mr. W. T. Halcrow's remarks, the Authors heartily ndorsed his expression of doubt as to the wisdom of present Parliamentary procedure, which permitted Bills of an engineering haracter to be rejected on their second reading without being xamined by a Select Committee. They were familiar with a recent ase in which a Bill embodying a hydro-electric scheme had been reated in what they considered was a most cavalier manner, and ad been thrown out by the House of Lords without the Promoters eing given the opportunity of putting forward their case or answering he objections that had been raised.

Mr. Halcrow referred to the alignment of the Carsfad dam. A careul study of all the factors, including rock-levels, foundation-conitions, and spillway-accommodation, had led to the adoption of the rrangement shown as representing the most economical compromise.

The Authors were interested to hear that on a canal in connexion vith one of the Grampian hydro-electric works similar trouble had een experienced with leakage through the canal-bank to that escribed in the Paper. They agreed with Mr. Halcrow that the rouble arose from the difficulty in making watertight a concrete lab laid in situ on a 1-in-$1\frac{1}{2}$ slope, and they felt that in future canals f a similar character special precautions should be taken to ensure thoroughly watertight lining.

With regard to the use of diesel locomotives in the Glenlee tunnel, t should be pointed out that they were not employed until after ll driving operations had been completed and a through ventilation f the tunnel assured. The precaution had been taken of fixing

Mr. Hunter.

special cleaning boxes to the locomotive-exhausts, and tests of samples of the air drawn from the tunnel where the locomotives were working had shown that those devices were effective in eliminating the harmful exhaust-gases. The forced ventilation provided had enabled time to be saved in blowing out the fumes after firing.

Messrs. Hawthorne and Williams.

Messrs. HAWTHORNE and WILLIAMS, in reply, stated that limitations of space had made it impossible for them to discuss in the Paper the considerations which settled the capacity of the reservoirs and head-ponds and the size of pipelines and machines. Mr. Williamson's remarks and data were a valuable addition to the descriptions of the Scheme, but a full exposition of the principles followed in its layout would require a complete Paper. Mr. Williamson had taken too literally the statement made in the Paper about the time Tongland power-station could run on full load with half-load inflow into the head-pond. That statement was made primarily to indicate one of several limitations beyond the Company's control. In point of fact the station had never been run under those conditions, but experience indicated that for economical operation 3 hours was the time that could be relied on.

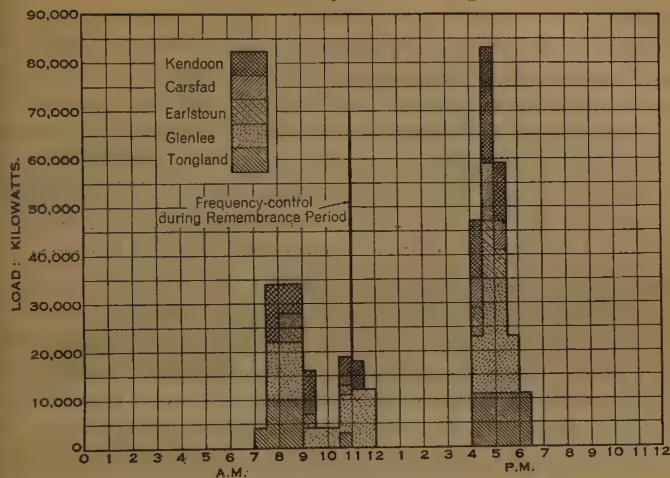
With regard to Mr. Williamson's remarks on the minimum quantity of water required for the fish-passes, it could be recorded that the passes were operating satisfactorily with an appreciably smaller quantity of water than that mentioned by him. The fishery experts now seemed to be quite satisfied, as a result of experiments and observation, that the fish would run more freely when the orifices were not fully drowned. Operating under those conditions, the cross walls were never flooded, and the risk of fish jumping out of the passes was practically negligible.

Mr. Coates appeared to suggest that it was more economical to burn coal in high-efficiency base-load stations than to utilize in the Galloway stations water which would otherwise run to waste. As the capital had already been expended on the hydro-electric stations there was no saving in capital charges if they were not run. There was, however, a saving in cost of fuel if they were run instead of steam stations. The advantage possessed by the hydro-electric stations in quick running-up and shutting-down and their use in controlling frequency were indicated in *Figs. 51* and *52*, which showed the output of the Galloway stations on the 11th November, 1937, throughout the day and during the Remembrance Period respectively.

Mr. Coates' surprise at the value of a hydro-electric scheme for frequency-control was difficult to understand. The question was that of trying not merely to maintain the frequency at 50 cycles per second under varying load-conditions, but, while doing so, also to maintain the maximum "thermal" efficiency of the system. O

Messrs. Hawthorne and Williams.

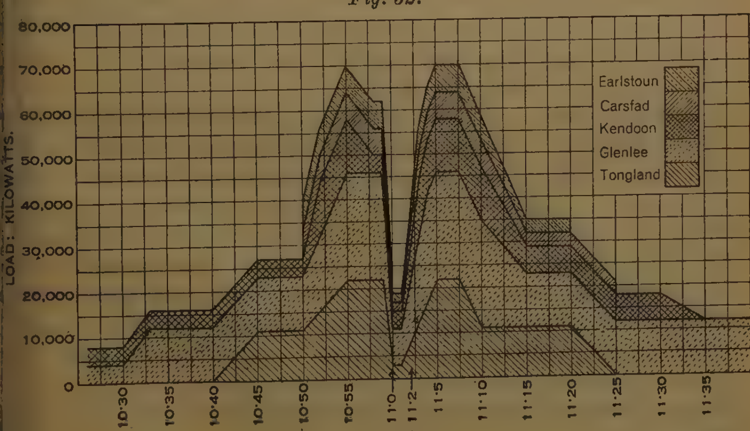
Fig. 51.



Output of Galloway Stations during the 11th November, 1937.

the Scottish Grid there were frequently during the winter comparatively rapid increases and decreases of load. To attempt to control frequency in those circumstances by varying the output of a base-load station would obviously prevent it being operated at its best efficiency and would also cause complications in the boiler-house. In hydro-electric stations such complications did not arise, and the practice therefore in such circumstances was to instruct the hydro-

Fig. 52.



Output of Galloway Stations during the Remembrance Period on the 11th November, 1937.

Messrs. Hawthorne and Williams.

electric station to vary its load with a view to maintaining constant frequency.

Several contributors to the Discussion had referred to the importance of the peak-load problem and had expressed the view that it would be possible for a long time to deal with the peak load by retaining in service the older and less efficient stations. The demand for power in Great Britain was increasing by at least 1 million kilowatts per annum, so it appeared that there would be plenty of work for the less efficient stations in dealing with the long-hour portion of the peak load, and that there would also be a necessity for stations which could economically deal with the upper and finer parts of the peak. In the opinion of the Authors it was doubtful whether it would ever pay to put in inefficient plant for the sake of keeping down capital expenditure, and also whether it would pay to use the sites of old stations for short-hour peak-load plant. Those stations, because of their situation in relation to the load and their site-facilities, would generally be of more value for dealing with the longer hours of the peak, leaving the shorter hours to be served by steam stations of highly-specialized design, by oil stations, or possibly by pumped-storage plants.

Mr. Mountain.

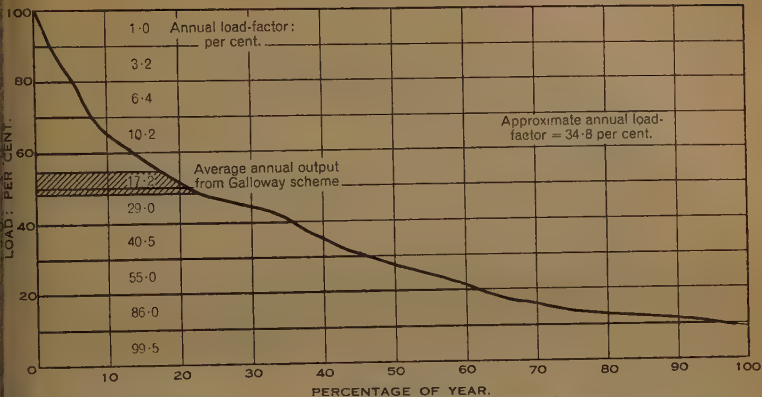
Mr. Mountain, in reply, observed that he was indebted to the President and to Mr. Coates for their support to the suggestion that a re-examination of water-power resources would be of value, to enable a comparison to be made with the cost of generation by steam stations. He also wished to thank Mr. Williamson for contributing to the Discussion a note of some of the essential factors which, in the consideration of hydro-electric schemes, were conducive to a low cost per kilowatt of installed plant when designed for peak-load operation and to thank Mr. Bridge for his statement of the conditions that would justify the development of water-power for the combined operation of steam and hydro-electric generation.

It had been mentioned in the Discussion that the Galloway scheme hardly fulfilled the requirements of a peak-load station, and *Fig. 53*, which was a typical load-duration curve, applied to the electrical requirements of Central Scotland and of North-West England, showed that the Galloway scheme did take an intermediate position between peak-load and base-load operation. The possibility of using obsolescent steam plant to generate the greater part of the peak load was not overlooked, and that method had been recently referred to at an informal meeting of The Institution, when the subject of electrical peak-loads had been discussed.

The comparison between the total cost of electricity from the Galloway hydro-electric scheme and the cost of electricity from existing steam generating-stations which had not been designed specially to deal with peak loads was given on p. 419. That com-

Mr. Mountain.

Fig. 53.

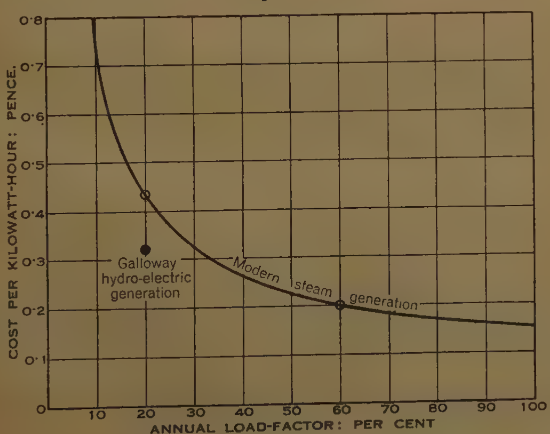


Typical Load-Duration Curve for Central Scotland and North-West England.

parison was shown in *Figs. 54* and *55*, firstly expressed as the cost of generation per kilowatt-hour and secondly expressed as the cost of generation per effective kilowatt-year. It was agreed that the cost of generation of electricity from a steam generating-station designed specially to deal with peak loads might be equal to or even less than the cost of electricity from the Galloway hydro-electric scheme.

The capacity of the 132-kilovolt transmission-line was dependent within certain limits on the power-factor of transmission, and from that point of view it was true to say that the addition of the Galloway generating plant increased the capacity of the transmission-line between Kilmarnock and Carlisle, by providing a source of reactive

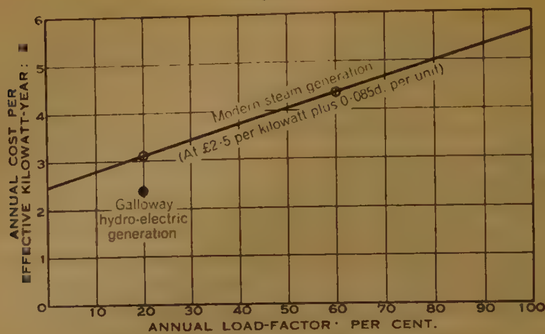
Fig. 54.



Cost of Generation of Electricity

Mr. Mountain.

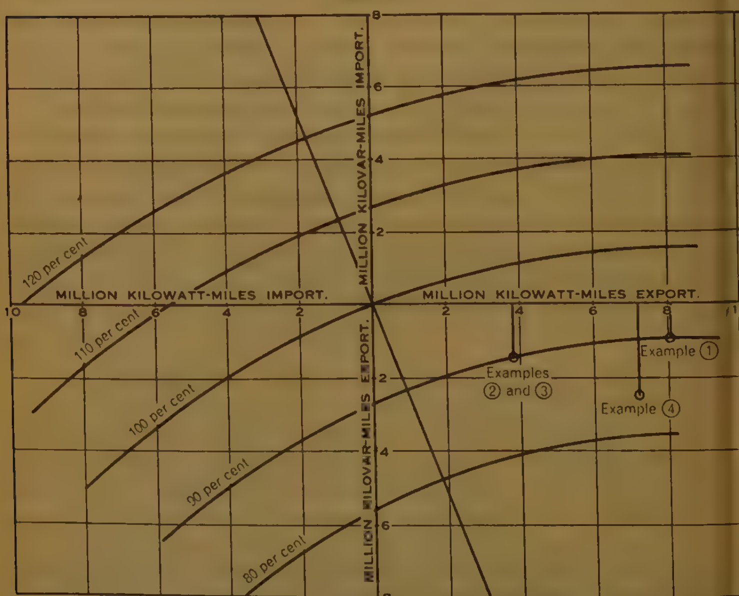
Fig. 55.



COST OF GENERATION OF ELECTRICITY.

kilovolt-amperes. *Fig. 56* showed the voltage-regulation for a 132-kilovolt single-circuit transmission-line, and illustrated the four examples given in Table XVII on p. 412.

Fig. 56.

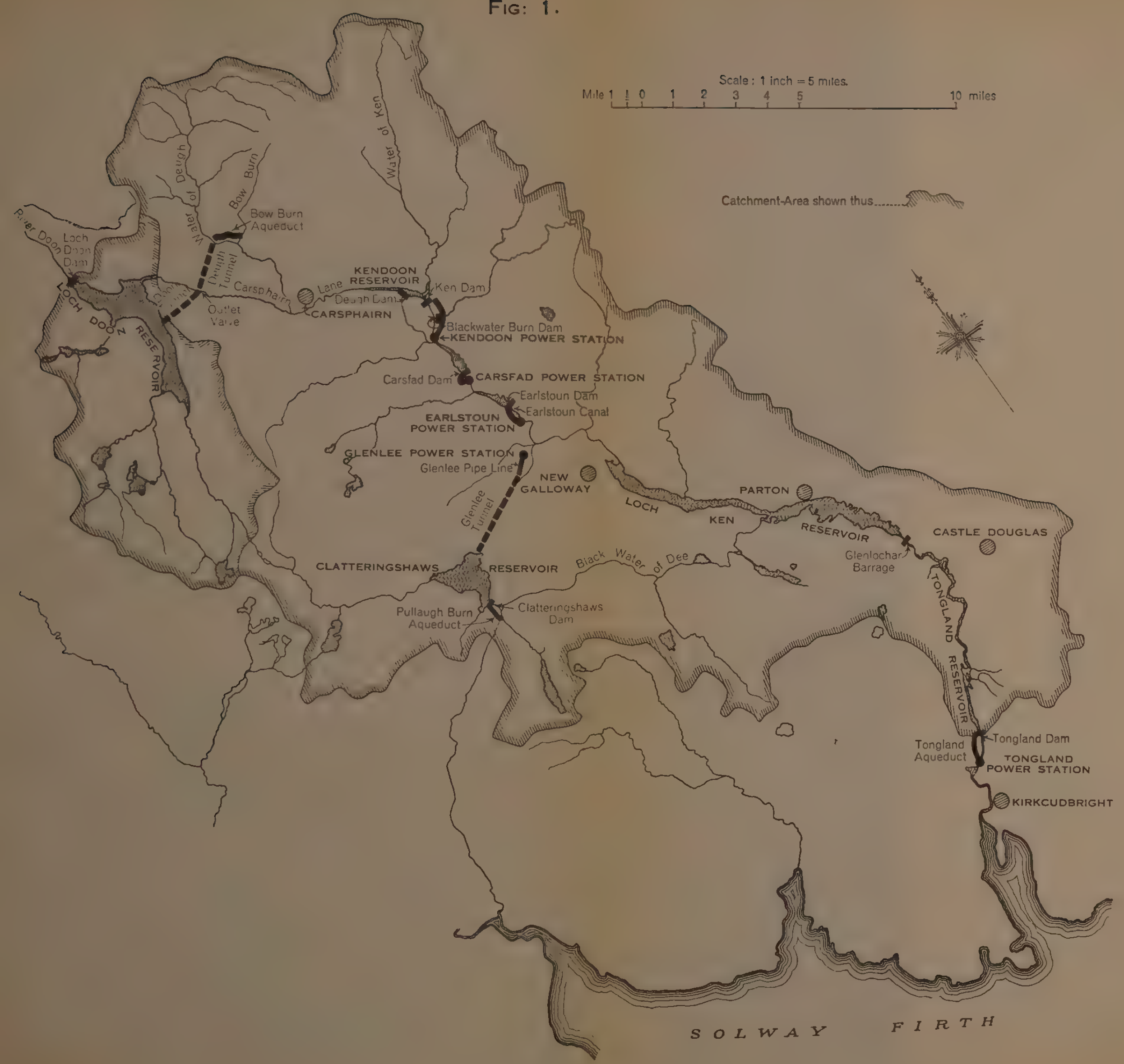


VOLTAGE-REGULATION FOR 132-KILOVOLT SINGLE-CIRCUIT LINE, DEFINED AS RATIO OF RECEIVING-END VOLTAGE TO SENDING-END VOLTAGE.

*** The Correspondence on the foregoing Papers will be published in the Institution Journal for October 1938.—SEC. INST. C.E.

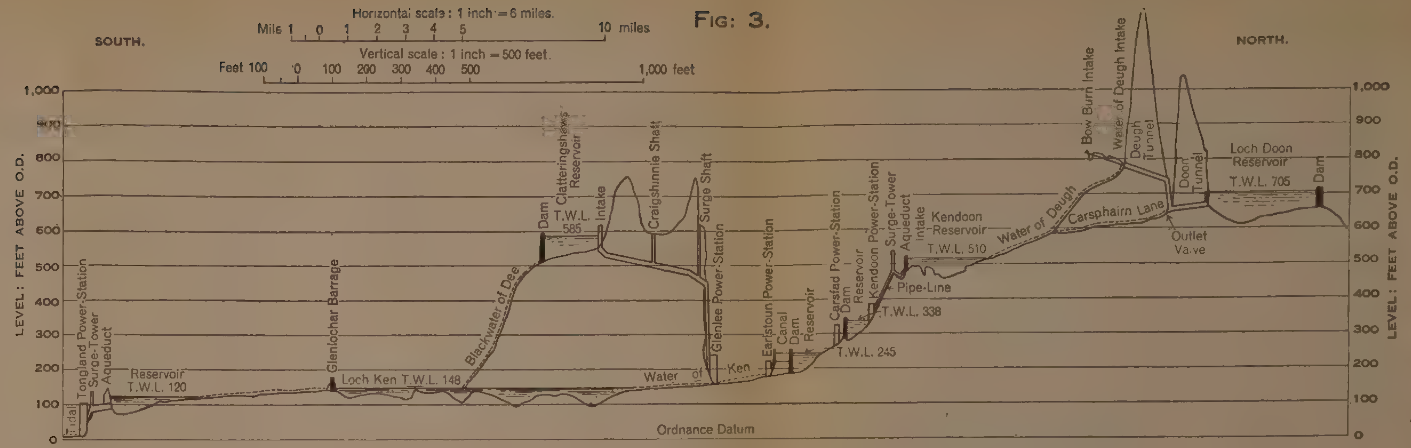
GALLOWAY HYDRO-ELECTRIC DEVELOPMENT: CONSTRUCTIONAL

Fig: 1.



KEY MAP.

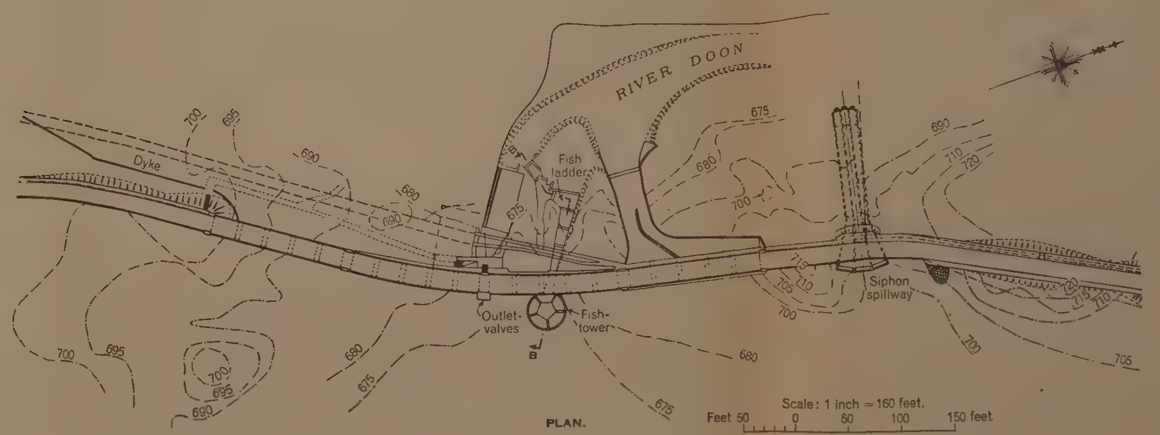
Fig: 3.



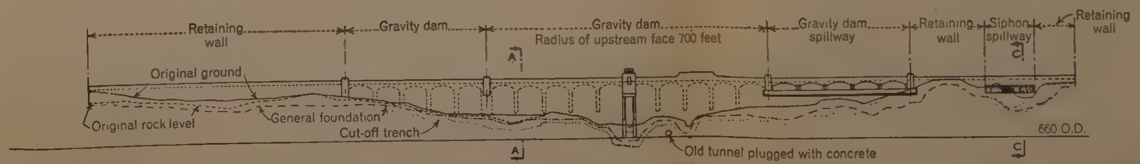
LONGITUDINAL SECTION FROM LOCH DOON RESERVOIR TO TONGLAND POWER-STATION.

PLAN

Figs: 4.

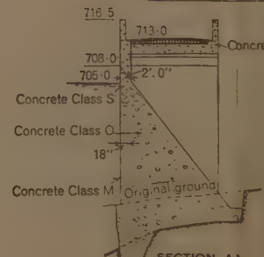


PLAN.

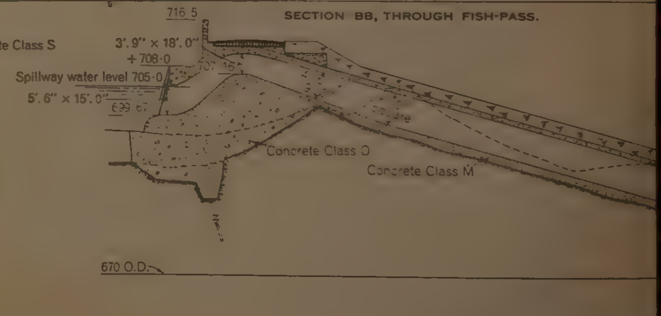


DEVELOPED UPSTREAM ELEVATION.

SECTION BB, THROUGH FISH-PASS.



SECTION AA.



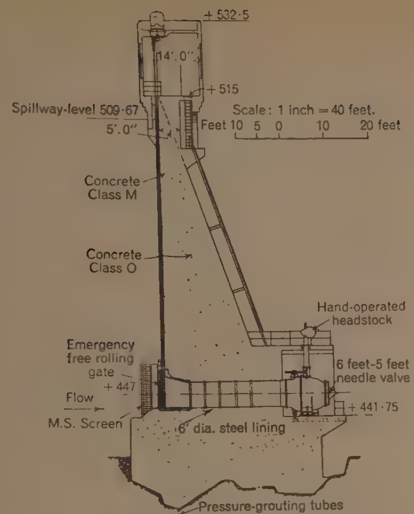
LOCH DOON DAM.

SECTION CC, THROUGH CENTRAL

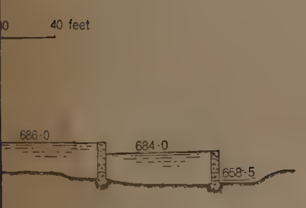
WORKS.

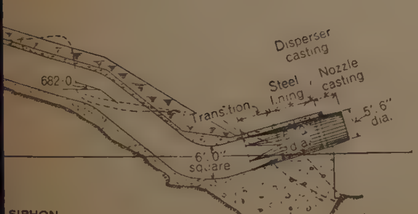
FIGS: 5.

DEUGH DAM.



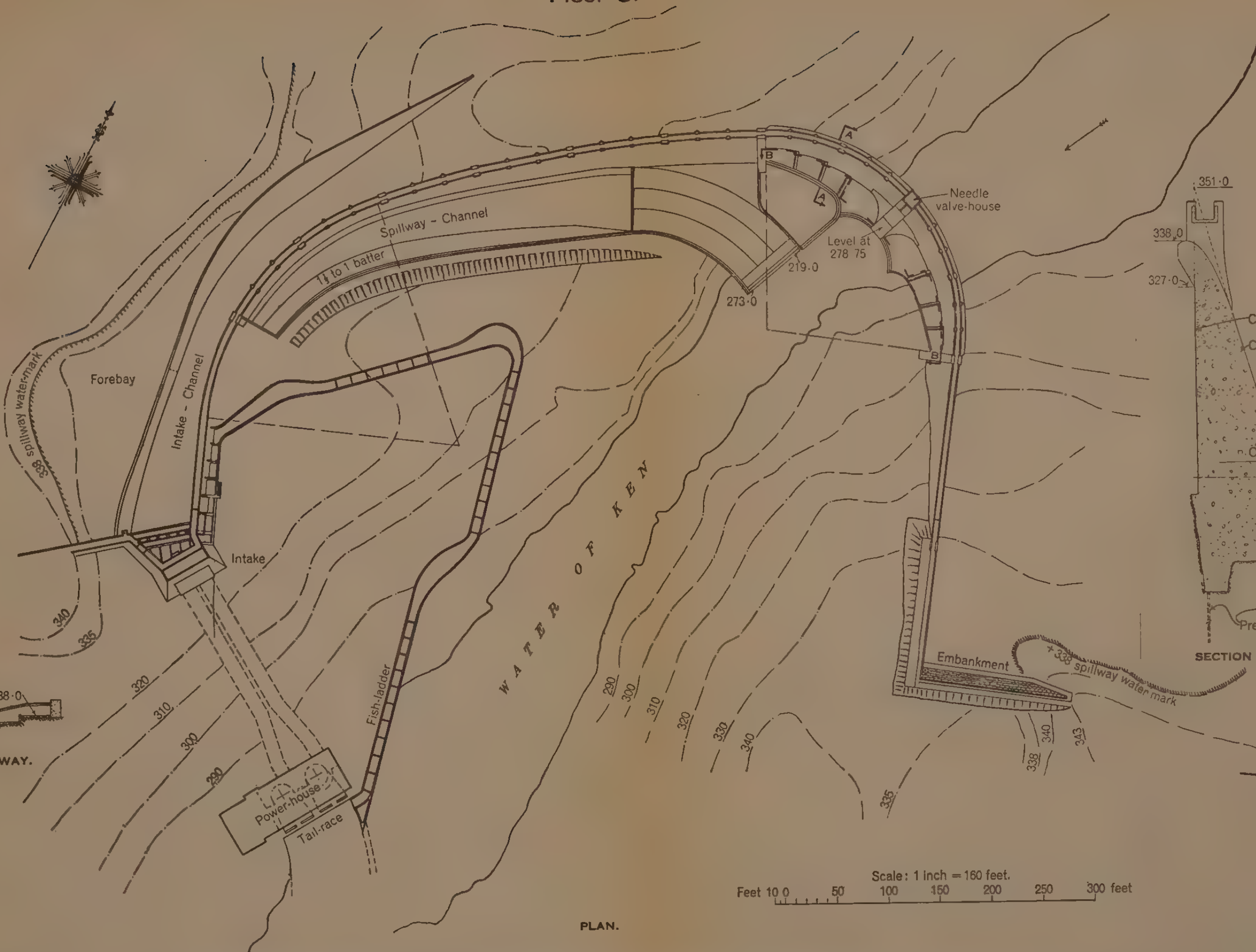
CROSS SECTION AT OUTLET-VALVE.



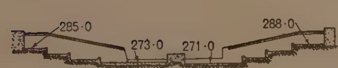


SIPHON.

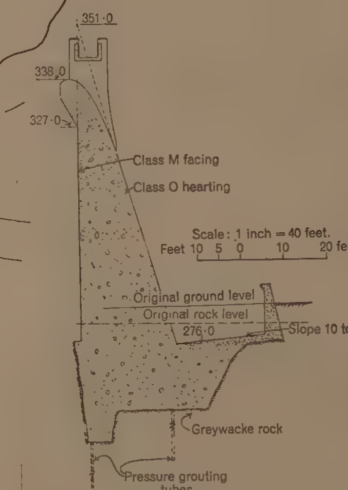
FIGS: 8.



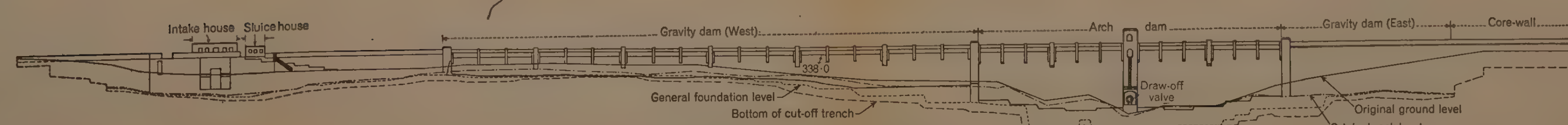
PLAN.



SECTION BB, THROUGH SPILLWAY.



SECTION AA, THROUGH ARCH DAM.

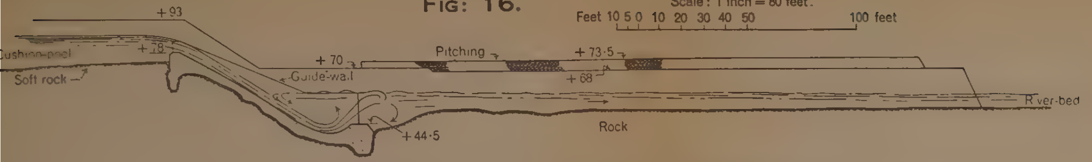


DEVELOPED ELEVATION, LOOKING UPSTREAM.

CARSFAD DAM.

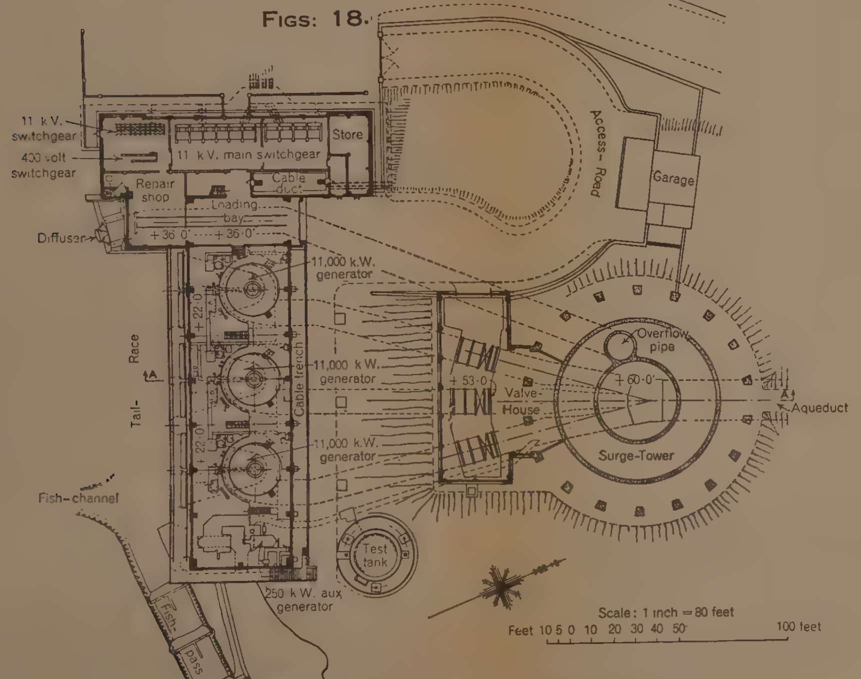
W. HUDSON and J. K. HUNTER.

FIG: 16.

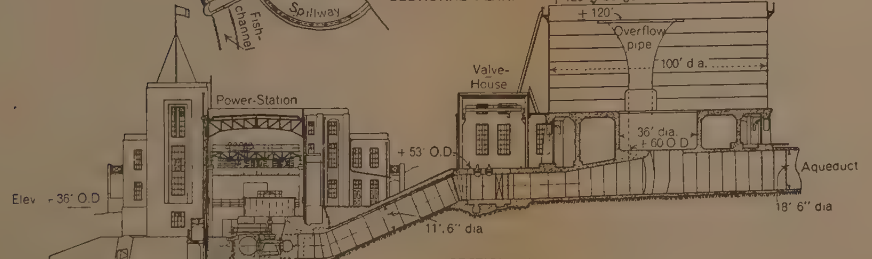


TONGLAND DAM: FLOOD-CHANNEL RECONSTRUCTION.

FIGS: 18.



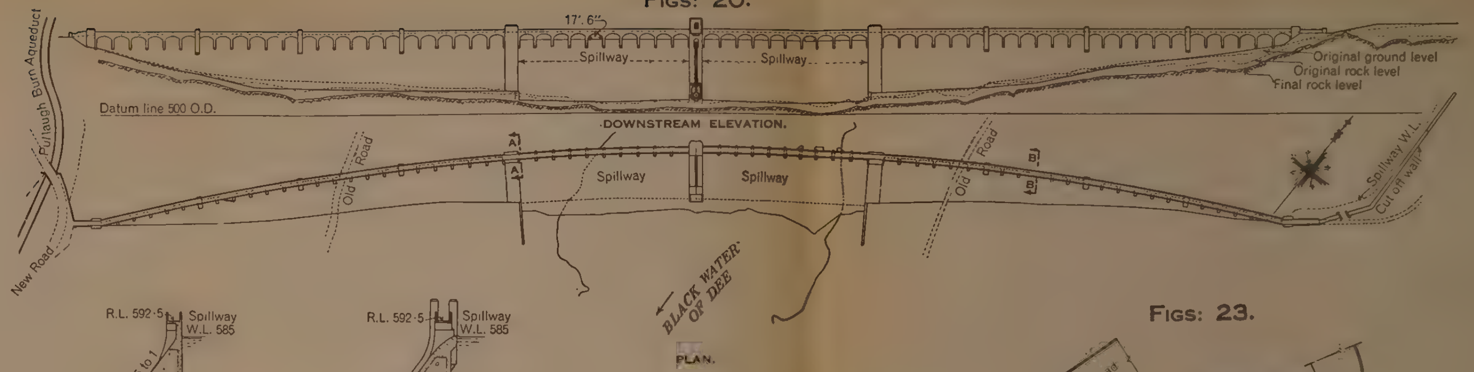
SECTIONAL PLAN.



SECTION AA

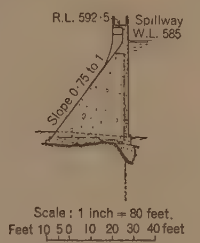
TONGLAND POWER-STATION.

FIGS: 20.

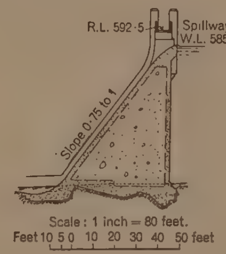


PLAN.

Scale: 1 inch = 200 feet.
Feet 50 0 50 100 150 200 feet



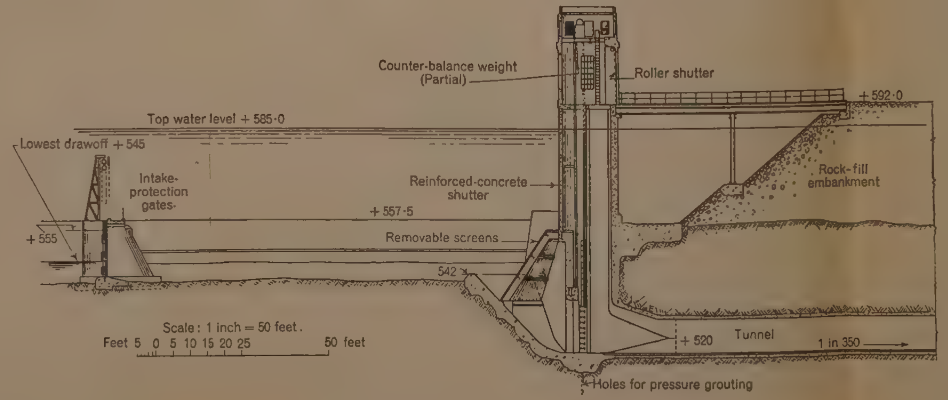
SECTION BB.



SECTION AA.

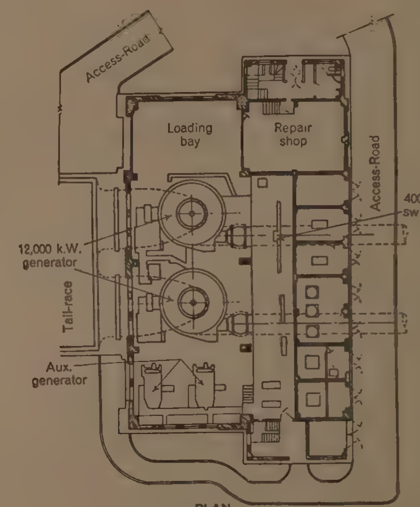
CLATTERINGSHAWS DAM.

FIG: 21.

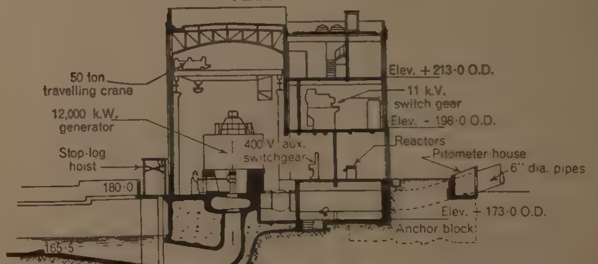


SECTION OF GLENLEE TUNNEL INTAKE.

FIGS: 23.



PLAN.



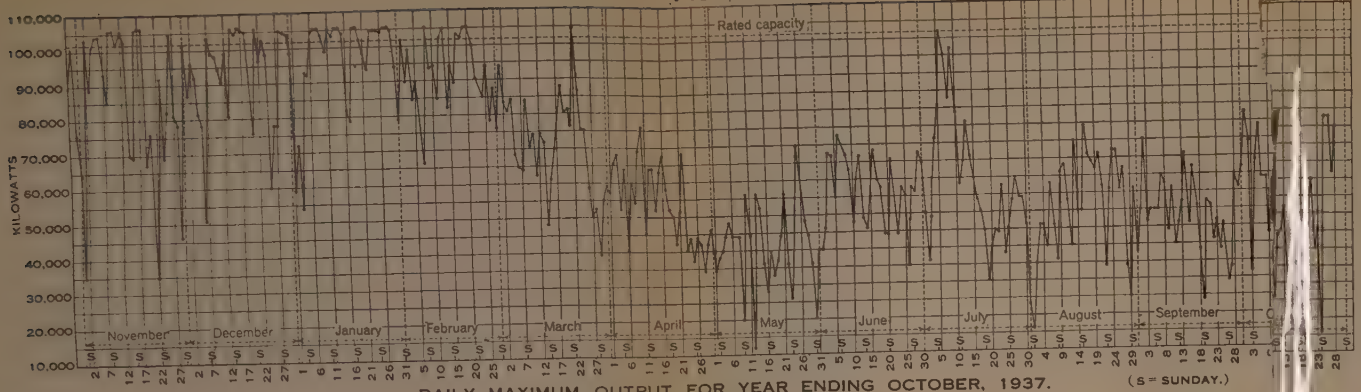
CROSS SECTION.

Scale: 1 inch = 50 feet.
Feet 10 5 0 10 20 30 40 50 feet

GLENLEE POWER-STATION.

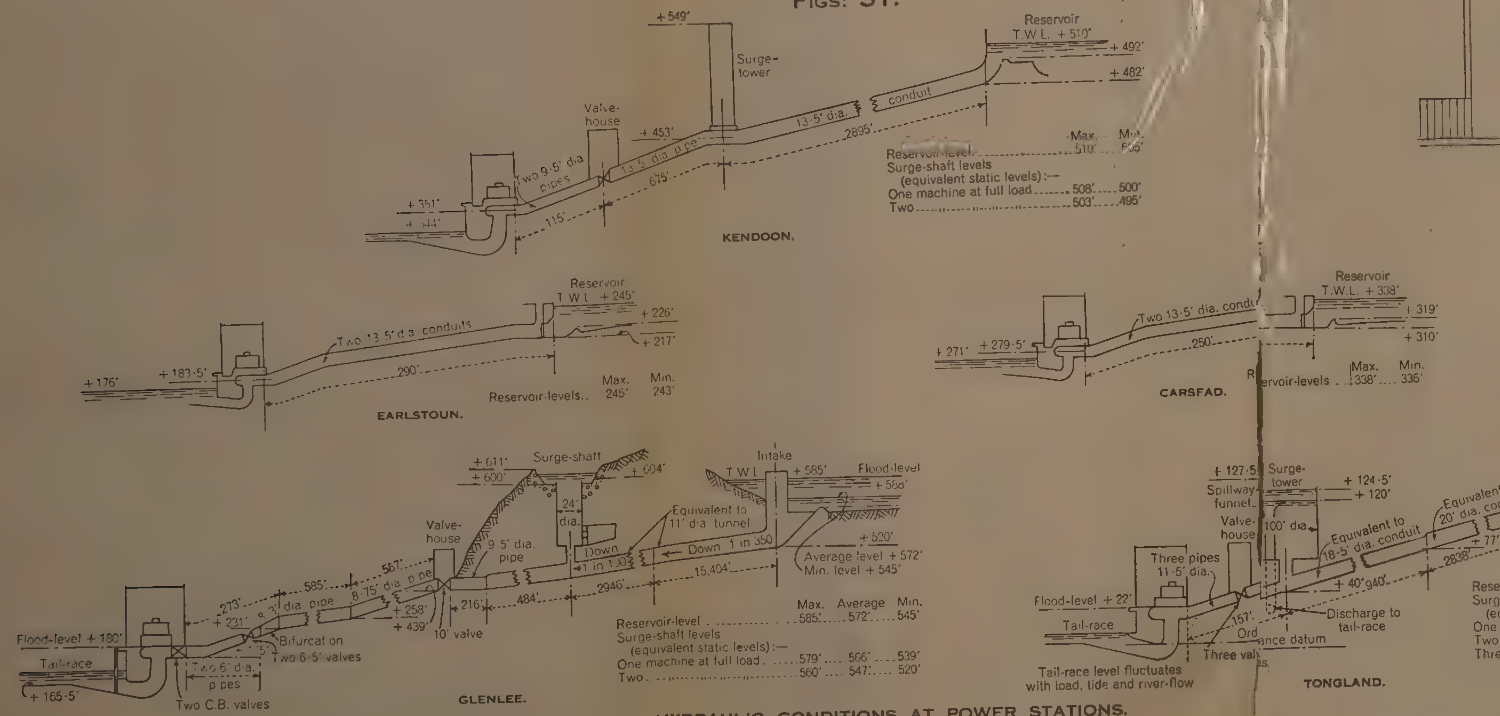
W. HUDSON and J. K. HUNTER.

FIG: 30.



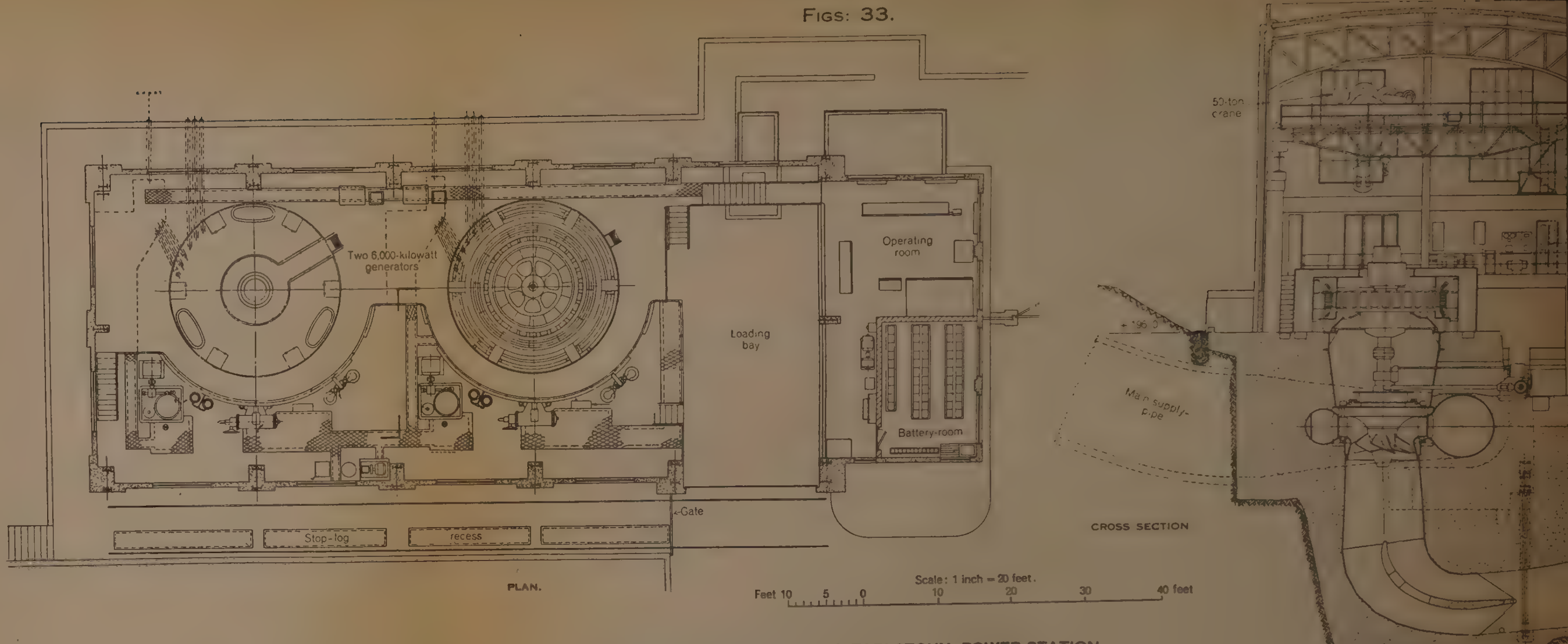
DAILY MAXIMUM OUTPUT FOR YEAR ENDING OCTOBER, 1937.

FIGS: 31.



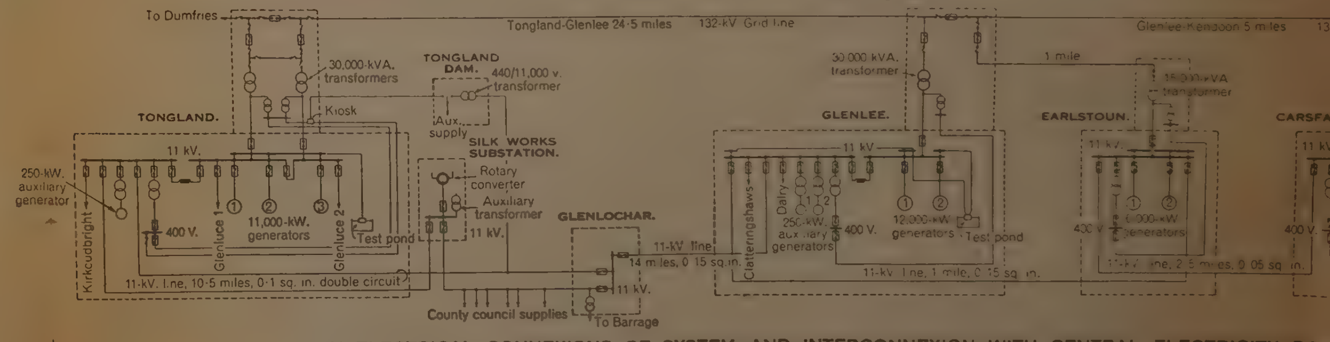
HYDRAULIC CONDITIONS AT POWER STATIONS.

FIGS: 33.



GENERAL ARRANGEMENT OF EARLSTOUN POWER-STATION.

FIG: 34.



MAIN ELECTRICAL CONNEXIONS OF SYSTEM, AND INTERCONNEXION WITH CENTRAL ELECTRICITY BOA

WILLIAM CLOWES & SONS, LIMITED: LONDON.

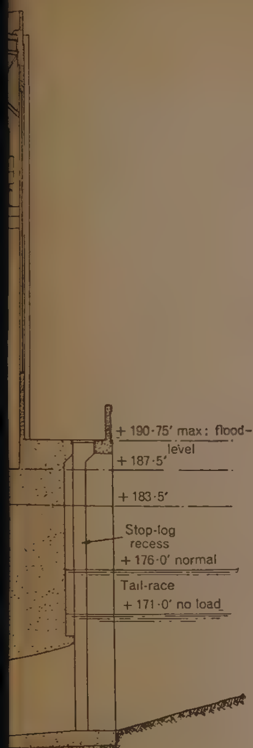
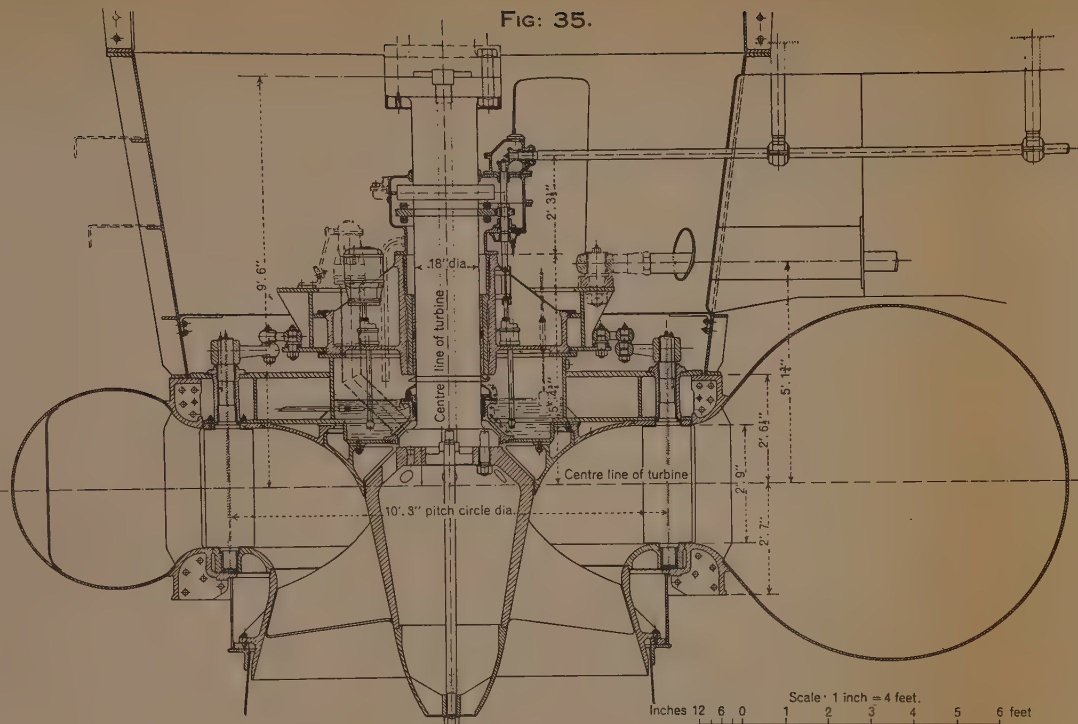
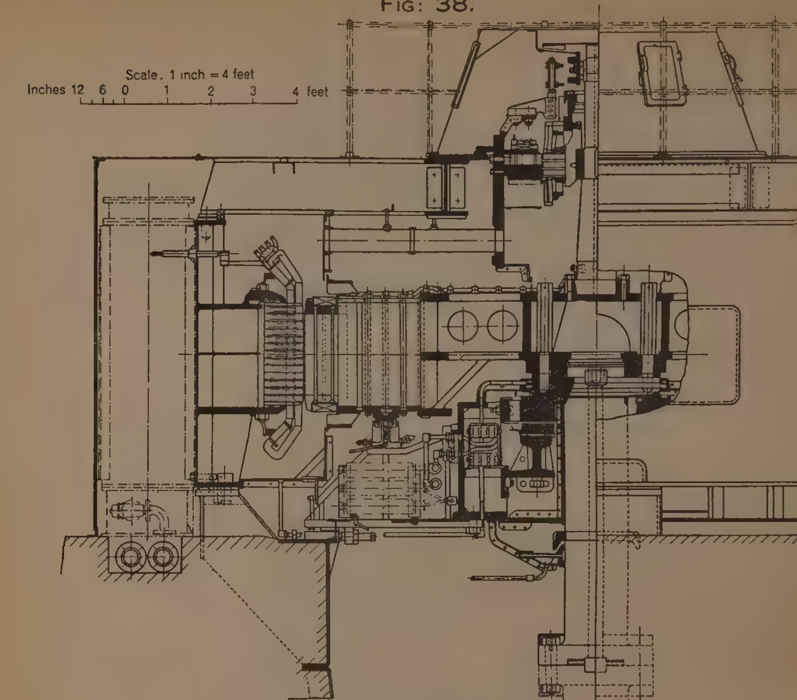


FIG: 35.

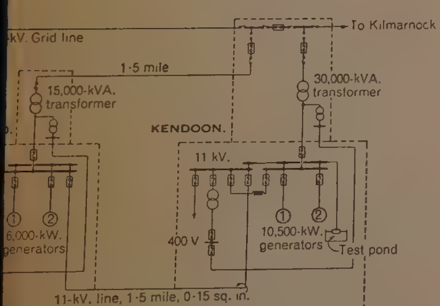


EARLSTOUN-CARSFAD TURBINE.

FIG: 38.

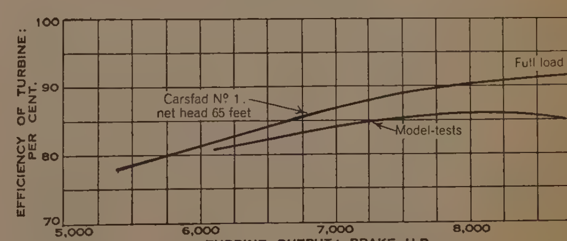


EARLSTOUN ALTERNATOR.



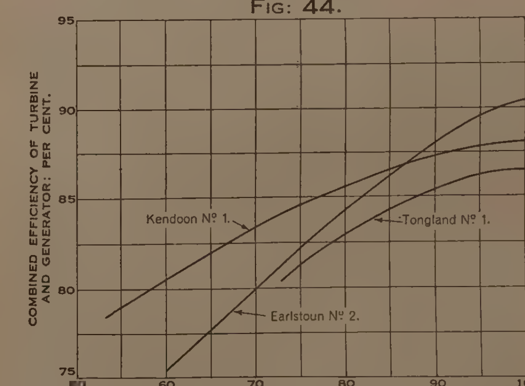
RD'S SYSTEM.

FIG: 36.



EFFICIENCY-CURVES OF MODEL AND FULL-SIZE RUNNERS FOR CARSFAD TURBINE.

FIG: 44.



OVERALL EFFICIENCY-CURVES OF GENERATING SETS.

W. HAWTHORNE and F. H. WILLIAMS.

ORDINARY MEETING.

8 March, 1938.

WILLIAM JAMES EAMES BINNIE, M.A., Vice-President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Member.

CHARLES ESMOND PARKINSON.

Associate Members.

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WILLIAM HERBERT AUSTIN, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
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ROBERT CUNNINGHAM BUCHANAN, B.Sc. (*Glas.*).
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George, Wallis, B.Sc. (*Witwatersrand*).
John Everard White.
James William Wilkinson.
Ion James Muirhead Williams, B.Sc. (*Cape Town*).
James Wilson.
Basil Claude Wealleans Wood, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.

The following Paper was submitted for discussion, and on the motion of the Chairman, the thanks of The Institution were accorded to the Author.

Paper No. 5170.

"Engineering Problems Associated with Clay, with Special Reference to Clay Slips." †

By Thomas Hardman Seaton, M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

Clays may be defined as rocks which are either plastic or can develop plasticity when wetted, their plastic properties being due in some cases to the development of specific clay minerals, but in other cases only to the extreme fineness of their particles.

Varieties of Clay.

Clays generally may be divided into four main groups, of which No. (4) in the following list is the most widely distributed :—

(1) China-clay and laterite.
(2) Loess.
(3) Boulder-clay.
(4) Sedimentary clays.

China-Clay and Laterite.—China-clay and laterite are clays formed "in situ", the former by hot-water decomposition of highly felspathic rocks, such as granite, with the formation of the so-called typical clay substance, kaolinite (hydrated silicate of alumina). Laterite results from the leaching action of hot moist surface-conditions on exposed basic rocks, and it consists mainly of alumina with iron oxide. (Bauxite is a variety of laterite.)

Loess.—Loess is a wind-borne fine dust of indefinite composition, though generally derived from glacial detritus, originating in mid-

† Correspondence on this Paper can be accepted until the 15th July, 1938.—Sec. Inst. C.E.

continental areas which are subject to constant strong winds. The areas in which deposition occurs are often semi-desert, and great thicknesses of the fine friable material may accumulate. In less arid conditions the fine deposit would be washed away by water.

Boulder-Clay.—Boulder-clay is glacier-transported material generally occurring in ground-moraines. It is essentially unweathered rock-flour, together with larger rock fragments produced by glacial grinding. It contains little of true clay-substances, and owes its plasticity to its rock-flour content.

Sedimentary Clays.—Sedimentary clays are compressed and partially dried-out muds, and are those with which engineers are generally concerned, particularly in the London area. The muds from which these clays are formed are the finer kinds of river-borne sediment, originating in the weathering denudation of rocks, and are deposited in water of moderate depth in lakes, lagoons and estuaries.

The mode of deposition of sediments is mainly that of the river depositing what it cannot carry in suspension because of its reduced speed ; thus, in the higher reaches of a river where the flow is fastest, pebble and gravel beds are formed. As the flow becomes slower, the deposits are graded down to fine sand, with finally silt and mud in its estuary where the flow is very sluggish. Another factor in this sedimentation is that the admixture of salt water causes precipitation of minute particles of colloidal size, previously held in suspension, by breaking down their mutual repulsion.

It may be of interest at this stage to trace the sequence of the various types of the whole group of what might be called the argillaceous or clayey sedimentary rocks. As pointed out previously, the origin of this group is river-borne muds, and the subsequent types of solid rocks are produced from these original muds by compression due to the increasing thickness and weight of the overlying strata.

The sequence is first mud, then clays, and then shales ; and, in somewhat unusual circumstances such as occur in seismic disturbances involving mountain-building movements of the earth's crust with lateral compression, slates may be formed.

The increasing hardness of these rock formations is due primarily to the compacting and the squeezing-out of the free water, with the result that when shale and slate are reached, there is practically no water left in the rock, although in these latter rocks the further compacting is due to the development of new minerals.

The fine mineral particles transported to these estuarial reaches are mainly quartz, felspars and micas, with a fair proportion of hydrous alumino-silicates (clay-substance) which are even more finely divided. In addition to these primary minerals, other minerals

may arise in the sediment, such as gypsum, calcite and dolomite by precipitation from sea-water; carbonaceous matter from marine organisms and plant debris; and pyrites and other iron sulphides from bacterial action in the mud. In well-compacted clays secondary mica (sericite) may develop at the expense of the various alumino-silicates present.

What is known as London clay is a sedimentary clay formed in this manner. A typical example, as freshly dug, was a stiff greenish-brown clay, with a damp feel, and tended to exude water in droplets; it contained 26·3 per cent. of water which was removed by drying at 105° C. An analysis of a dried sample is :—

	Per cent.
Silica	54·1
Alumina	20·5
Ferric oxide	4·3
Ferrous oxide	2·7
Iron sulphide	1·3
Lime	1·7
Magnesia	3·2
Alkalies as Na_2O & K_2O, etc. (by difference)	4·3
Titanic acid	0·9
Sulphates (SO_3)	0·2
Carbonates (CO_2)	2·8
Carbon	0·2
Combined water	3·8
	100·0

From these figures, together with various physical tests, it can be calculated that the mineral composition of the dried clay would be approximately as follows :—

	Per cent.
Quartz	26·0
Felspar and mica	28·0*
Kaolins	21·0
Easily soluble ferro-magnesium minerals	15·4
Limonite	2·0
Calcium carbonate	2·5
Magnesium carbonate	3·3
Gypsum	0·3
Pyrites	1·3
Organic matter	0·2
	100·0

* Possibly, felspar, 12 per cent.; mica, 16 per cent.

The principal mineral constituents of clay are quartz, felspar, mica and clay-substance. *Quartz* is chemically all silica. It is the main

constituent of sand and is a hard crystalline substance insoluble in water. *Felspars* are chemically alumino-silicates of soda or potash, or soda and lime. They are minor constituents of sand and of about the same hardness as quartz, but decompose under weathering action more readily than quartz. *Micas* are complex hydrous alumino-silicates with well-developed cleavage; there are three main types, (*a*) biotite, containing iron oxide and magnesia, (*b*) phlogopite, containing magnesia, and (*c*) muscovite, a potash mica. They are fairly soft minerals, and their fragments in sediments are of a platy or flaky character. *Clay-substance*, kaolin, a hydrated aluminium silicate, is the typical mineral constituent of china-clay, and to some extent is present in all clays. It is produced by the decomposition of the felspars and occurs in very small plates. It is believed that in most clays the clay-minerals present differ from kaolin in containing, besides alumina, silica and water, varying proportions of iron oxides and alkalies. Most of them, moreover, show no crystal form but are amorphous gels.

The quartz, felspars and micas could possibly be identified in clay under a powerful microscope, but it is doubtful if the most powerful instrument could discern the matrix, so finely grained is it. This moist matrix has the properties of a stiff jelly, and in unaltered clay the particles of colloidal dimensions are packed close together, and there is surface-attraction between them, making the mass rigid and strong. When clay becomes penetrated by excess water, the effect is to remove the particles to greater distances apart and reduce their attraction and the consequent rigidity of the mass.

The Author suggests that the fact of clay having a colloidal character cannot be too strongly emphasized, as it must, in his opinion, have first consideration in suggesting measures to maintain the stability of clay soils.

In practice, so far as it concerns the properties of sedimentary clays, namely, compacted muds deposited in estuaries, it means that the colloidal particles of small dimensions can be precipitated by neutralization of their electrical charge by the agency of an electrolyte, leaving practically solid masses, yet with certain flow-properties (plasticity) resulting from the small dimensions of the ultimate colloidal particles.

Mud is changed to clay by its compacting and partial drying-out, together with the squeezing-out of excess water and the packing together of the fine particles. Since a large proportion of the particles are of platy habit (the micas and kaolins notably so) and some proportion is of colloidal dimensions, the adhesion between the particles is strong and is aided by the presence of a certain amount of water. A good analogy to clay would be a block of stiff jelly

in which 25 to 40 per cent. of fine mineral particles has been disseminated. Such a material would, on absorption of a certain amount of water, become an unstable jelly.

In its undisturbed state, therefore, clay has a fairly strong and stable structure, and is almost impervious to water. Exposure to erosive agents, however, such as free water, frost, or heat-changes involving alterations in water-content, causes the structure to become unstable by enabling more water to penetrate between the particles to an extent sufficient to overcome the adhesion between them. Such excess water, in effect, acts as a lubricant and the particles slip over each other when any stress is applied to the clay. It follows, therefore, that to have a strong clay-structure the water-content must be kept at its normal figure. If, however, an unstable clay were allowed to dry out it would again become stable (though possibly developing shrinkage-cracks) and remain so if protected against weathering and excess water.

The term "clay," as the civil engineer knows it, is applied generally to brown clay. Deep-seated undisturbed clays are often of bluish-grey colour; London clay is of this type. In any blue clay, the change of colour to brown is an index of approaching instability; this colour-change, which is due to the oxidation of the small amount of iron sulphide present to iron hydroxide, is an indication that aerated water has penetrated the clay, giving a more unstable structure due to the breakdown of the adhesion between adjacent particles. Most of the surface works discussed in this Paper have been in such brown clay.

Surface-Drainage of Clay Soils.

The most common problem connected with clay is the surface-drainage of clay soils. It is evident, having regard to its characteristics, that clay must not be permitted to become too dry or too wet. Open ditches in clay soils are, therefore, inadvisable if the clay is exposed, as in dry weather it cracks, allowing water to penetrate and disintegrate it. This feature, it is suggested, cannot be over-emphasized, as the cause of slips in clay cuttings has at times been traced to the presence of an open ditch at the top. Almost as unsatisfactory are pipe-drains with the trench filled to the surface with very large broken stone or hard-core, as the large interstices permit, to an appreciable extent, similar weathering action to take place on the sides of the trench and cause the filling material to become choked, with the result that the drain ceases to function efficiently.

Where the circumstances demand that expense shall be kept to a minimum, shallow drains, suitably graded and filled with fine

ashes, are particularly effective, but where a fair amount of water will require to be dealt with and the drains will be on an appreciable gradient, it is advisable that pipe-drains should be laid. These may be straight-jointed in instances where they are not subject to pressure—particularly side pressure. Where, however, the drain has to withstand pressure, socketed pipes should always be used so as to preserve the continuity of the drain should any local deformation take place.

The pipes should be carefully bedded into the clay for the lower third of the diameter, the trench being filled with flints, broken stone, or hardcore, and topped with fine material. The trench should be at least three times the diameter of the pipe, with a minimum width of 2 feet where there is a possibility of the sides of the trench squeezing in and tending to choke the filling material. The best filling material has been found to be 3-inch flints. Broken stone and hardcore, especially if on the large size, often become choked in time, whereas flints appear to be gradually compacted but remain unchoked.

Probably the most difficult drains to maintain are those at the foot of railway-cuttings in clay. These, being subject to heavy adjacent pressure, are particularly liable to be forced out of alignment and level. Experience suggests that the invert-level of permanent-way drains in clay cuttings should not be less than 3 feet below rail-level and that they should be bedded on concrete. In connexion with such drains the importance of an adequate number of inspection-manholes cannot be over-stated. These should not be at greater distances apart than 150 feet, should be formed with a sump below the invert of the pipe and, if of brick, should have the vertical joints left open above the level of the top of the pipe. They should be of such a size that drain-rods can be easily used in them, 4 feet long and at least 2 feet wide internally being a convenient size ; the covers should also be removable by one man.

Road and Railway Construction on Clay Soils.

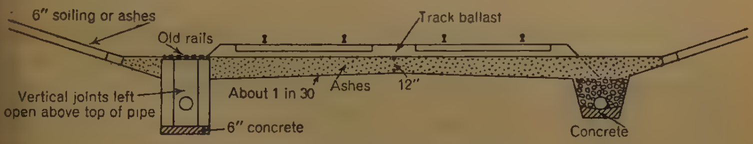
The characteristics of clay soils suggest that for a first-class road it will be found that the most economical foundation on clay is concrete, reinforced as considered necessary after taking into consideration the pressure the clay will withstand and the loads the road-surface will have to bear. Roads of secondary importance might be constructed with a compacted layer of 6 inches of ashes on a suitably graded and drained formation, followed by 9 inches of hardcore or rough stone, topped and graded with a layer of good binding stone or slag, and surfaced with tarred macadam. They should be bounded by a substantial concrete curb and edging

founded below the clay formation, though even in this case it is possible that a concrete road or foundation would be justified and more economical unless ashes, hardcore, or stone are readily available at a cheap rate.

In connexion with the construction of railways under like conditions, it is a point for consideration whether, for a new line to carry a heavy traffic, the first cost of providing a continuous concrete foundation would not be justified by the saving in future maintenance of the track. The foundation might take the form of a mattress of weak concrete under a 12-inch layer of ashes and ballast.

If the concrete foundation were placed at a higher level, say 6 inches below the sleepers, it would require to be of appreciable thickness and suitably reinforced to withstand the loads, particularly the impact of rail-traffic at high speeds. If, however, it is considered that the expense of the foregoing cannot be justified, it is suggested that a compacted layer of 12 inches of fine ashes should

Fig. 1.



Railway in Clay Cutting

be put down on a drained formation graded to about 1 in 30, the top ballast being 1½-inch granite, rough stone, or slag. This, however, should not be put down until the ashes have become thoroughly consolidated under traffic, as shown in *Fig. 1.*

If the top ballast is put down on a layer of ashes of insufficient thickness or before these have become consolidated, the ballast is driven through the ashes into the formation, which is then penetrated by water and softened, and the clay is forced up to the surface between the sleepers and at the sides of the track. A greater thickness of ashes than 12 inches is unnecessary and inadvisable, as it is found that this material does not usually compact to a greater depth.

Drains laid at the side of the track should be placed on a substantial bed of concrete, so as to assist in preventing the spreading of the clay formation and the consequent distortion of this and the drain.

The maintenance of permanent way on a clay formation should be by shovel-packing. If beater or any form of percussion-packing is used, there is danger that the formation will be damaged and the track eventually become unstable.

Treatment of Clay Slips.

It is perhaps not generally realized what a large amount of money is spent annually by railway-companies alone in remedying clay slips. On the Author's late district—which was perhaps the most notorious in Great Britain in this respect—the expenditure was and is of the order of £17,000 per annum, most of which is spent on sections totalling 65 miles only.

Clay slips are usually caused by an alteration in the normal water-content of the clay. In the case of clay cuttings this may be due to the clay drying during hot weather, during which surface-cracks form and sometimes reach to a depth of 8 feet below ground. If the clay is in this state and rain falls at a greater rate than the clay can absorb, its structure breaks down and become unstable. It is, therefore, of paramount importance that these cracks should be filled in with fine material and rammed so as to check the drying-out of the clay, and also to stop the ingress of too much water.

One of the most effective means of preventing the clay cracking is to cover its exposed surface with a layer of fine ashes, 6 inches at least in thickness, which may be soiled if suitable material is available. If a less thickness than this is laid down, heavy rain is not absorbed with the result that the ashes may be washed down the slope and the clay exposed.

Reference has already been made to the practice of making open ditches at the top to intercept the ground-water as one source of slips in clay cuttings. Broken or defective drains may also be a cause, or the free discharge of field drains into the cutting. The making of ponds by adjoining owners has been known to make trouble in this direction. The lowering and widening of cesses, and taking away the toe of the cutting or its becoming softened by inadequate or defective drainage, are other causes.

When a slip has occurred, it is recommended that at the outset an immediate search should be made of the undisturbed ground behind the slip and that any surface cracks should be filled in so as to prevent the ingress of water. In certain cases the construction of a wall to retain the slip and prevent its extension is unavoidable. Walls for this purpose must generally be of massive construction and are necessarily expensive.

It is not proposed in this Paper to discuss in detail the design of walls to retain clay slips, but it might be mentioned in passing that it is important in the construction of such a wall to take the foundation well down into the undisturbed earth below the slip. With regard to the drainage of the wall, the Author favours dividing the

wall into sections by expansion-joints of an elastic compound, each section being drained by means of a longitudinal pipe-drain on a set-off as low as possible at the back of the wall and under a good thickness of dry backing. The outlet should be taken directly into a drain as far as possible in front of the wall, so as to ensure that water does not inadvertently get into the foundations and cause sliding to take place. Where movement of a retaining wall is occurring, this can usually be arrested by the building of counterforts behind or in front of the wall, the counterforts to be well bonded into the wall, and those behind tied also into the main mass by tension-rods or cramps.

It is suggested, however, that wherever possible the construction of a wall should be avoided, as, unless the drainage and support of the slip are also undertaken, there is always a possibility of trouble at a later date by the wall being subjected to greater pressure than it was designed to withstand. The slip may also be deeper-seated than was apparent, and further movement may take place below the original slip.

The stabilizing of a slip can usually be undertaken by drainage and by giving it support. The first step is to provide means, possibly of a temporary character only at first, of draining away any water that may be met with in the slip. A trench is then cut through the centre of the slip so as to ascertain the depth of the displacement, to release any imprisoned water, and to provide means to take away any that may drain into it in future. The width of the trench will depend on the assumed greatest depth of the slip. A usual width is 4 feet 6 inches, but where the slip is much broken up and believed to be deep, and where extra support and working space will probably be required, it may be advisable to increase the width to at least 6 feet. The depth having been found, arrangements should now be made to lay a permanent main drain to take away any water that may continue to percolate into the slip. If this is along the foot of the slip, the drain should be laid on a substantial bed of concrete and the drainage-trench well packed with dry filling so as to act as a support to the toe of the slip. If in a cutting, it will probably be found that at the top the line of the slip approaches the vertical, and in this case the top should be trimmed back to a slope of 1 in 3. If, however, the limitations of the site prevent this being done, it will be necessary to give the undisturbed ground support by counterforts and by packing some of the disturbed earth against the face.

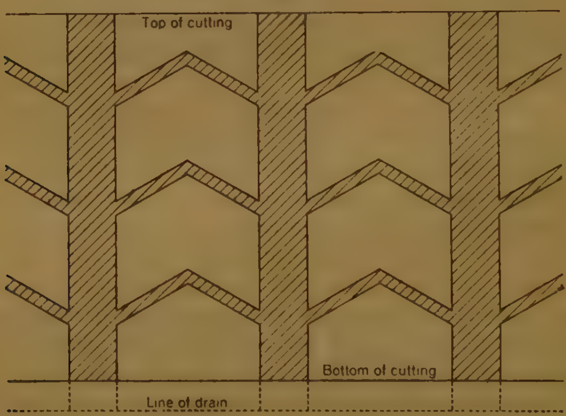
The bottom of the trench should be taken below the slip, dished, and an open-jointed drain laid on it, the trench being packed with hardcore up to the line decided on as the finished surface of the cutting, and the top layer of hardcore being broken small. Where

the sliding surface of the slip is appreciably inclined, the bottom of the trench should be benched.

When the bottom of the slip is below the level at which a main drain can conveniently be laid in, the excavation for the counterforts should still be taken below the slip and filled with weak concrete up to the level at which it can be drained.

The hardcore-packed and drained trench functions as a counterfort, giving support to the slip by the friction of its sides. It also assists in stabilizing the displaced clay by draining away any water which may accumulate. The number of counterforts to be provided and their distances apart depend on the extent and depth of the slip. In difficult cases it is usually necessary to make them a

Fig. 2.



distance apart equal to three times their width. The Author was both interested and gratified to discover a few years ago that Robert Stephenson had, in 1840, adopted on the London and Birmingham Railway with success a similar method in dealing with cutting-slips and had in fact used the same spacing apart of the counterforts, making the counterforts 5 feet in width and 15 feet apart. He, too, regarded the slip as a mass of material moving down an inclined plane, and proposed to counteract this by the friction of the slip against the sides of the counterforts. The drain at the bottom was, however, omitted.

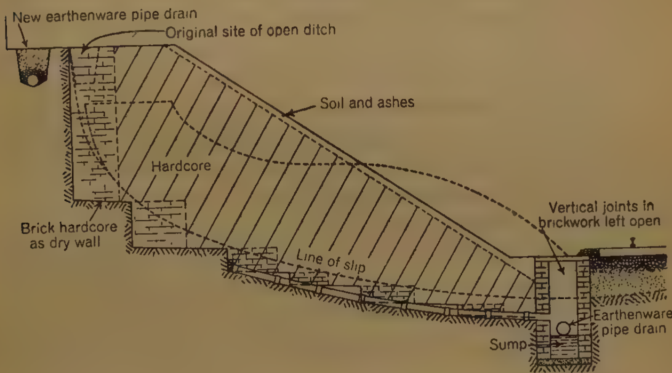
In passing it might be mentioned that experiments have been made with trenches at various angles, but, whilst some of these are quite effective, the difficulty in timbering makes this method more costly than normal trenches which are generally effective. Very unstable material may, however, require additional surface-support

which can be given effectively by a "chevron" system of counterforts between the parallel ones, as shown in *Fig. 2*.

The new slope having been decided upon, the excavated earth can be packed to some extent between the counterforts and act as support to the top portion of the cutting, if this cannot be trimmed back. It is important that the surface of the finished slip should be covered with a layer of ashes at least 6 inches thick to prevent further weathering action taking place on the clay, and if good soil is available to spread over this, it is an advantage. *Fig. 3* is a diagram of a cutting-slip and shows the remedial measures to be adopted.

Clay slips have at times been dealt with by burning. This consists of mixing with the clay that has slipped a quantity of small

Fig. 3.



Clay Slip in Cutting.

coal and burning the whole. Sometimes the resulting clinker is fairly large, hard and not very absorbent; water, therefore, quickly passes through it to the bottom of the slip, and unless means of draining this away are provided, further trouble may occur. Often, however, the clinker is soft and disintegrates under weathering action. It does not bind well together, and as the back of the slip may approach the vertical little support is given to this. There is a possibility also of contraction-cracks occurring in the undisturbed clay during the burning owing to the evaporation of the water normally present in clay, with the result that the entrance of water into these may cause further disintegration and movement in the underlying clay. This method is, therefore, to be avoided.

In connexion with slips on clay railway-embankments, there is little doubt that the removal of the gravel and ash ballast and the substitution of stone or slag were the cause of slips, in that they per-

mitted the free access of water into the underlying clay. Where the track was lifted on the old ballast the trouble was minimized, but in many cases it was not possible to raise the rail-level.

Slips on embankments are often due to the original ground-surface becoming waterlogged and softened. Where embankments are formed on sidelong ground, a permanent improvement can be made by laying a drain at the foot of the embankment on the high side so as to intercept the ground-water and prevent its percolation into and under the embankment.

Slips at times occur with unfortunate frequency where embankments have been widened. When this is done, as in the case of new embankments, it is imperative that the original ground should be well drained; otherwise the formation of the embankment may be softened and slips occur. Care should also be taken in forming the embankment, particularly if the formation is weak and liable to settle, when some strengthening of the formation may be necessary. In any case the embankment should be formed in layers, each of which should be thoroughly consolidated before the next layer is laid down, with the surface of each layer falling to the outside of the embankment. The maximum thickness of each layer should be 10 feet. If the formation is weak, the thickness should be less.

The practice of tipping embankments the full height and wider at the top than the finished width, and then of dragging down to the required slope, cannot be too strongly deprecated, especially in clay soils, as the tipped material is not properly consolidated and its repeated disturbance alters the natural water-content, resulting in initial instability.

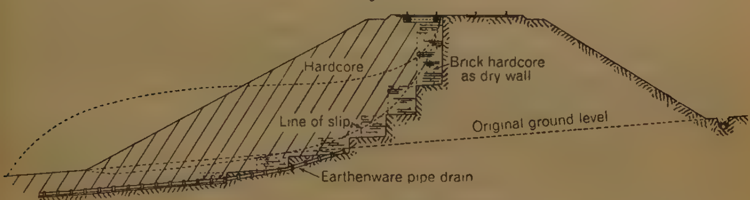
When an embankment is widened the existing slopes should be benched, and, instead of the traditional large benches at intervals, the Author favours a series of small benches. Ash-filled drains should be made through the widened embankment at intervals so as to take away any water that may collect on the old cess. Where also the tipped material is somewhat unstable, it is advisable to form a toe of large broken stone or hardcore at the foot of the embankment, and to work ashes into the surface of each layer.

Slips occur at times in clay railway-embankments at bridges and culverts, especially where the original ground-level is falling along the centre-line of the railway. In such cases an underbridge forms a bulkhead and intercepts the ground-water, and, unless adequate measures are adopted to drain the water away, slips will occur. It is recommended for bridges and culverts through embankments that a longitudinal pipe-drain be placed behind the abutments and side-walls under a good thickness of dry backing and below the original ground-level. The bridges themselves in these situations

often become defective, and, in particular, the abutments of those with a girder superstructure move forward at the top and require to be rebuilt or strutted. The movement is always greatest at the left-hand side, namely, in the direction of rail-traffic. In every case where movement has occurred no dry backing has been found behind the abutment. Actually it was the prevalence of this movement that caused the Author to make his researches into the structure of clay, as he formed the conclusion that he was dealing with a material akin to a jelly.

It is suggested that in connexion with bridges and culverts in clay embankments the traditional wing-walls at an angle of 45 degrees to the abutments are unwise, as it is rare that such a wing-wall of any age is not fractured or distorted due, apparently, to the clay having a wedging action. In such cases it is recommended that as far as possible the wing-walls should be kept in line with the abutments. It might be mentioned here that in one case

Fig. 4.



Clay Slip on Embankment.

where a thick pier split vertically, the pier was found to have a solid core of clay.

At the commencement of a slip in a railway embankment it is often an advantage, where space is available, to load it with heavy material, such as gravel or discarded track-ballast, so as to drive out and stabilize temporarily the moving mass as soon as possible. When movement has been arrested, the procedure is generally the same as for slips in cuttings, though possibly the packed trenches need not be at such frequent intervals. The trenches should, however, be taken to the back of the slip, the track being supported on a timber frame as necessary. Ashes and soil are again a necessity for protecting the new slope of the embankment. *Fig. 4* is a diagram of a typical embankment-slip, and shows the remedial measures.

Examples of Slips.

It will now perhaps be of interest to mention a few characteristic examples of slips and the remedial measures adopted to deal with

them. The first example refers to the efficacy, where space is available, of re-forming a clay cutting to a critical slope and protecting the surface. This method was adopted some 17 years ago to the top portion of a large cutting-slip, the back of which was almost vertical. The undisturbed earth was trimmed back to a slope of 3 to 1, the surface covered with 6 inches of ashes and a piped surface-water drain laid in to cut off the ground-water. The result has been entirely satisfactory, whereas similar places in the vicinity not so dealt with have continued to break away and slip. A number of cuttings in the same district have subsequently been cut to this slope, and up to the present no slips have taken place, whereas when a steeper slope has been given slips have occurred.

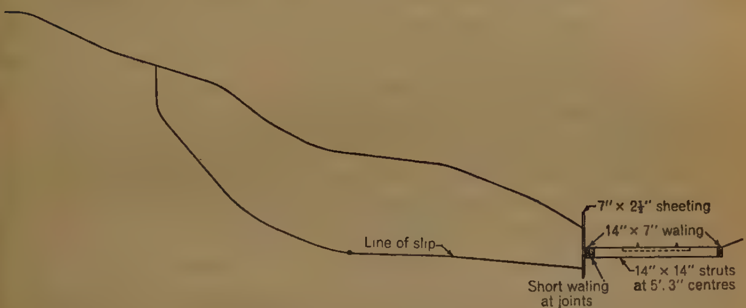
A typical example of a slip in a railway-cutting was one which occurred on the Southminster branch of the London & North Eastern Railway, a single line of railway in Essex. Here the cutting has a greatest depth of 30 feet, and shallow slips had occurred some years ago. The method adopted at that time was to trim back the toe, to cut shallow drainage-trenches through the slips at intervals and roughly to fill these with hardcore overlaid with ashes, the excavated clay being heaped up between the trenches. It may be mentioned that some of the worst slips with which the Author has had to deal have been those where this method has been adopted, as the displaced material is usually in a state of unstable equilibrium, and, when it begins to slip, moves rapidly and exerts considerable pressure.

In this instance the first indication was a slight lifting and slewing of the track. On this being reported, an immediate inspection of the site was made, and it was decided that there was a danger of the line becoming blocked if support were not given to the toe of the cutting. Timber was quickly taken to the site and the slip strutted temporarily as shown in *Fig. 5*. 7-inch by $2\frac{1}{2}$-inch sheeting was placed vertically at the toe, in front of which was a 14-inch by 7-inch horizontal waling with another short one in front of the joints. The waling was then strutted by 14-inch by 14-inch timbers at 5-foot 3-inch centres placed between the sleepers of the track and wedged against a continuous 14-inch by 7-inch waling at the foot of the cutting on the other side of the track. The strutting saved the situation, and as a measure of the pressure exerted by the slip it may be observed that the 14-inch by 14-inch struts were forced more than an inch into the walings in several cases.

The position having being made secure temporarily, steps were then taken to stabilize the cutting by cutting drainage-trenches 6 feet wide through the slip, which were spaced 18 feet apart and packed with hardcore, and by trimming back the slope to 1 in 3. On

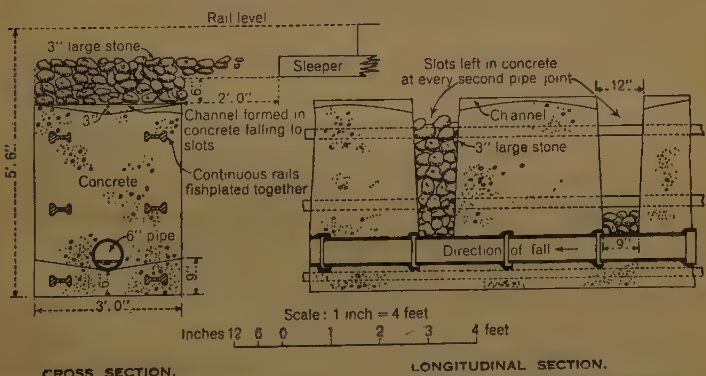
excavating the trenches the clay in the slip was found to be much broken up, several slips being found, whilst the preceding dry years had formed fissures some 8 feet in depth in the clay in places. The unstable character of the material generally and minor slips necessitated in some places counterforts of a "chevron" pattern between

Fig. 5.



the main trenches. During the cutting of the trenches opportunity was taken to record the sliding-surface of the slip; this is also shown in *Fig. 5*. The slip extended over 3 feet below rail-level, and a method of supporting the toe was adopted that had proved successful in similar instances.

Fig. 6.



Concrete Sub-wall with Drain at Foot of Cutting Slip.

The permanent drain to be laid alongside the track to take the water away from it and the drainage-trenches was concreted-in as shown in *Fig. 6*, openings being left at the joints of the pipes for the ingress of water, whilst the continuity of the concrete around and above the pipes was obtained by reinforcing it longitudinally

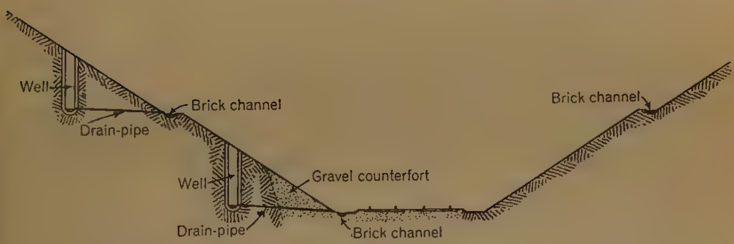
by old rails. The reinforced-concrete beam thus formed has been found to be a satisfactory method of holding the toe of a heavy slip of this character. One advantage of this method over a retaining wall is that, should there be a forward movement, it is below the track and does not affect the clearances.

The third example refers to an embankment-slip extending over ½ mile. The embankment had been tipped on sidelong ground ; it supported a double railway-track carrying a heavy main-line traffic and had been unstable for a number of years. A wet season, followed by a heavy fall of snow, was succeeded by a rapid thaw. Shortly after this a length of embankment on the low side commenced to move and the track subsided faster than it could be packed up. On taking over the remedial work the Author had trenches cut into the embankment and found that a basin had been formed in the original ground-surface which was badly waterlogged. The basin was drained permanently by pipe-drains discharging into a nearby stream, and rubble counterforts were constructed to give support to the undisturbed portion of the embankment, measures being taken to prevent the continued percolation of water under it. These consisted of pipe-drains at the foot of the embankment on the high side. In laying the drains a considerable volume of water, which had undoubtedly been finding its way under the embankment, was met with in places. The measures taken were entirely successful, and the bank became more stable than for many years past though subject to a heavy passenger and goods traffic, the former attaining high speeds.

Though not strictly in a clay formation, it will perhaps be of interest to refer, in conclusion, to a cutting which at the time of its construction nearly 100 years ago achieved a somewhat unenviable notoriety, namely, that taking the former Eastern Counties Railway through Brentwood hill, Essex. This cutting, which is 60 feet in depth, is driven through what are known geologically as the Bagshot beds. The London clay here passes upward into the Bagshot sand and there is no marked plane of separation. The sand and gravel deposits are probably post-glacial ; they are irregular in structure, and the sand in places contains water. Considerable difficulty was experienced in making the cutting owing to the nature of the sand and silt, from which the water could not be separated, the tendency being for the silt to flow away with the water. Large slips occurred and the work was almost at a standstill until an ingenious scheme of well-drainage was evolved. This took the form of an upper and lower series of wells in the wettest part of the cutting, the upper series discharging into an open brick-channel on a benching half-way up the slope. A gravel counterfort was

provided to support the toe of the slope and also to hold back the silt while permitting the free discharge of water. The wells were steined as in ordinary well-work. Each well had an outlet-pipe discharging into an open drain. The system proved very successful, the only trouble that occurred subsequently being due to the partial collapse of some of the wells which blocked the outlets.

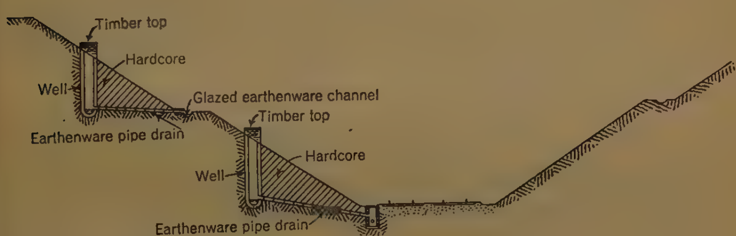
Fig. 7.



BRENTWOOD CUTTING, ESSEX, IN 1840.

The wells became surcharged with water and burst, and local slips were caused. It fell to the lot of the Author to make good the slips and recondition the drainage-system. In so doing the wells were brought up above ground-level and fitted with wooden covers which prevented animals, etc. from falling into the wells and blocking the outlets. The covers also obviated the considerable expense occasioned in cleaning out the wells.

Fig. 8.



BRENTWOOD CUTTING, ESSEX, IN 1930.

Fig. 7 shows the cutting as originally drained in 1840, and *Fig. 8* the reconditioning carried out in 1930. This cutting is referred to in Volume III of the Minutes of Proceedings.[1]

[1] C. H. Gregory, "On Railway Cuttings and Embankments; with an account of some Slips in the London Clay, on the line of the London and Croydon Railway." Minutes of Proceedings Inst. C.E., vol. iii (1844), p. 135.

In this Paper the Author has endeavoured to investigate generally the character of clay, to ascertain its tendencies, and to formulate methods for overcoming its idiosyncrasies. The methods suggested appear to be confirmed by experience in actual cases, having regard to the characteristics of the material. It is realized, however, that it is only possible to lay down general principles and not hard-and-fast rules, as the factors in each case must be treated on their merits.

Acknowledgements.

In conclusion, the Author would like to record his indebtedness to Mr. R. J. M. Inglis, M. Inst. C.E., Engineer, Southern Area, L. & N.E. Railway, for facilities granted in connexion with the preparation of this Paper, and to the assistance he has received in his researches from Messrs. John Hill and Charles Walker of the Chemists' Department of the L. & N.E. Railway; also to Sir William Bragg, President of the Royal Society, for helpful advice.

The Paper is accompanied by eight drawings, from which the Figures in the text have been prepared.

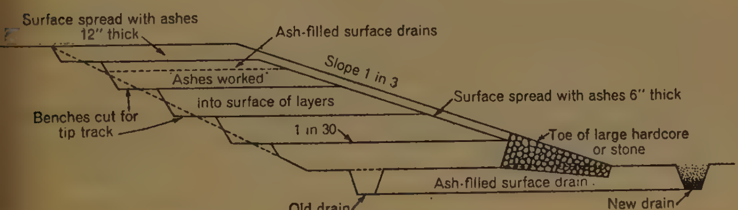
Discussion.

The Author.

The Author exhibited a number of lantern-slides illustrating his 'aper. He explained that the Paper had been written with a twofold bject, firstly, in the hope that young engineers would find it useful s a record of a somewhat varied experience in dealing with clay, nd, secondly and more importantly, with a view to stimulating esearch into the structure of clay. He called special attention to he colloidal character of clay, and to the impairment of its stability f the normal water-content were exceeded.

An important point about the drainage-manholes shown in *Fig. 1* p. 463) was that such manholes were often covered with enormous labs of natural or artificial stone which required several men to lift hem. He would never use a manhole-cover too heavy to be lifted y one man, as otherwise there was a tendency for the cleaning of nanholes to be neglected, resulting in the drains becoming silted up. t was most important that the manholes should be kept clear of silt.

Fig. 9.



Widening of Embankment with Clay Soil.

Fig. 9 showed a suggested method for widening embankments in lay soils. The Author stated that he knew from experience that rouble often occurred in connexion with widened embankments, and he attached great importance to the drainage of the surface on which the embankment was formed. Where the material was unstable a rubble toe such as was shown was of great value. In the north of England, where large blocks of stone were available as quarry-refuse, he had held slips by massing blocks at the toe. It would be noticed from *Fig. 9* that ashes were worked into the surface of each layer; where the clay was very sticky that had been found to be advantageous.

At one cutting on the Woodford and Ilford line counterforts had been constructed obliquely. The clay slips in the cutting had been

The Author. burned some years previously. The burnt clay was on the move the whole time, which made timbering difficult, especially for the obli counterforts; for that reason he did not recommend that form counterfort, as it did not appear to be more effective than vertical form.

Modern cuttings in clay were often taken out to a slope of 1 in he had never known such a cutting give way or show any signs slipping, provided that the clay had been properly protected agai weathering action.

Fig. 10 showed a typical slip at an underbridge in a clay emba ment; he suggested that the bridge had been acting as a bulkh and holding up the ground-water, with the result that the format on which the embankment had been tipped had become soften resulting in a slip. He had known many such instances.

He had mentioned on p. 469 that the abutments of girder und bridges sometimes moved forward and broke away from the win necessitating complete rebuilding. Bridge-engineers were n taking the loads on abutments well back from the face, and it v advisable that that should be done, as with the ordinary flat b plate there was often a great deal of weight on the face of the w which tended to encourage the type of failure mentioned.

Clay slips were responsible for many cases of so-called coa erosion. During many centuries land had continually been l from the east coast of England, and particularly between the Thar and the Wash. Cities and harbours had disappeared and mou of rivers had silted up. In many cases trade and industry had l the districts affected. Valuable land and property was at pres seriously menaced, and the general position was now so grave th remedial measures would seem to be a matter of national importan Recently there had been extensive cliff falls at Cromer, Horsey h been inundated, the sea-defences at Lowestoft had been breach and road access to an important aerodrome was seriously threaten The sea-walls in the river Crouch had been breached, and seve thousand acres flooded at high water. The situation at Frinton-o Sea, shown in *Fig. 11*, was most disquieting. In that case as many others, erosion by the sea was a minor circumstance; the cl of the cliffs gradually broke away, became disintegrated, and slipp Fortunately the prevailing wind during winter was northerly a during the summer southerly, but occasionally wave-action in hea gales removed great quantities of fallen material from the toe of t slips. Attempts had been made to stem the destruction, but w only temporary success, and they would serve to make the ultima catastrophe of greater magnitude. The most important problem w to maintain the stability of the soil, and the longer that remedi measures were put off the more costly they would become.

Fig. 10.

BRIDGE NEAR SHENFIELD DAMAGED BY SLIP.

Fig. 11.

SLIPS IN CLAY CLIFFS AT FRINTON-ON-SEA.

Fig. 12.

CONCRETE COUNTERFORTS NEAR MOTSPUR PARK, SOUTHERN RAILWAY.

Fig. 13.

FOLKESTONE WARREN AFTER SLIP IN 1915.

Fig. 14.

FORESHORE, RETAINING WALL, AND GROYNES AT FOLKESTONE WARREN, AFTER SLIP IN 1937.

ustrate that natural slips were not always due to erosion, he would ention that a hillside, also in Essex, miles away from the sea, had arted to come down on its own, slipping in just the same way as e embankments and cuttings to which he had referred. The Author.

Mr. F. E. WENTWORTH-SHEILDS remarked that the Paper had been ritten at the suggestion of The Institution's Research Sub-Committee on Earth-Pressures. It was many years since the particular pect of soil-mechanics in question had been discussed at a meeting f The Institution, and they felt that it would interest all engineers ho had to do with the stability of earth. The remedy of slips such s those discussed by the Author was a problem that had not yet een completely solved. A Paper such as that under discussion erved to draw attention to the very fine work that was being done y the Building Research Station in the investigation of slips such s those described, and in the question generally of estimating what ne stability of an earth structure really was. That work was a iece of real engineering research, in that it was not only a fundaiental research into the properties of soils but also an endeavour to pply science to the practical problems of the engineer. The amount f money spent on that research was trifling, he ventured to say, ompared to the amount that was wasted on account of present morance of the subject, and the Building Research Station had so ar been handicapped by want of funds and of staff. He felt that it vas of the greatest importance not only that the Building Research tation should be supported in its work on the subject, but also that he work should be extended to all the college laboratories in the Jnited Kingdom, so that an engineer could have his soils readily ested by a local laboratory, and, moreover, that young engineers night be taught how to carry out such work and how to apply their nowledge to economic design. Mr. Wentworth-Sheilds.

The Committee had realized that the questions with which the uthor dealt would make the subject of an admirable Paper and hould provoke an interesting and helpful discussion. The Paper rought out the facts that clay could either be a very good and table foundation or backing or a very bad one, and also that the roperties of a clay could be varied within very wide limits, and ould be immensely improved by works such as had been described n the Paper, designed to reduce its water-content. A few years ago t Bremerhaven he had been greatly interested to see that wells and rains were being sunk behind one of the big quay-walls. As there vas no outlet for the drain into a cutting alongside, the wells had o be pumped out; the amount of water taken from them was small, ut it was claimed that the mere fact of removing that trifling amount f water had an immense effect on the stability of the clay. Many roblems of a similar nature called for research in soil-mechanics,

Mr. Wentworth-Sheilds.

and it had to be admitted that Great Britain was behind certain ot countries in that study. It was for The Institution of Civil Engine to see that that position was remedied.

Dr. Stradling.

Dr. R. E. STRADLING remarked that, though he himself was no specialist on soil-mechanics, he had a group of colleagues at Building Research Station who were working on that subject, wh was of great importance and interested him very much. He his colleagues were very grateful to the Author for bringing forw his practical experience, which would be of the greatest help to one working on the subject. At the same time, he had to ackno ledge a feeling of disappointment with regard to pp. 457 to of the Paper, as the outlook there displayed took no notice at of the vast amount of work that had been done on soil-mechanics Professor Terzaghi and his followers, to which engineers in ot countries were paying great attention. He ventured to suggest t the composition, structure, and characteristics of clay as portra in that introductory section were not those which mattered to engineer, apart from the Author's emphasis on the question moisture-content, which was universally known to be of v importance with the type of material in question. The work Terzaghi and others abroad, and to some extent the work of Building Research Station in England, had emphasized again a again that it was possible to take samples of clay and other so to examine them in the laboratory, and from the data so obtained carry out a design of an earthwork founded to some extent at le on reasonable data and with a mathematical basis.

He did not wish to infer for a moment that the design of earthwo would ever be reduced merely to mathematics, because he did believe that that would be the case; the practical experience of engineer was of vital importance in such work. It was, howev in his estimation a tremendous step forward that it should established, at any rate to some extent, on a fundamentally sou basis, though that did not seem to have attracted the attention the engineering profession in England to the extent which it had other countries. For two reasons in particular, which he wo discuss, he was very concerned about that fact. The handling earth and the building of earth structures was one of the old branches of engineering, so that the engineer dealing with it was m concerned with his practical experience and was rightly proud of In dealing with matters that depended very largely on practi experience, great care had to be taken to allow for any changes the conditions on which that practical experience had been bas Engineers, however, were now seriously altering those conditio chiefly by the introduction of modern mechanical methods handling earth. Firstly, the time of consolidation could be reduc

enormously by modern methods that it was wrong, he would Dr. Stradling. ggest, to argue that because it had been possible to build an em-nkment on a certain site by the methods of 30 years ago it would equally possible to build a bank on the same site by modern ethods. Secondly, modern methods provided a much higher gree of consolidation. Most engineers at any rate would consider at to obtain consolidation was the principal desideratum in ilding an embankment, for instance. He thought that that was rticularly true with the older methods of building, but recent ork had indicated that modern methods of mechanical handling ight in certain circumstances produce excessive compaction. ne very interesting suggestion had been made [1] that the consolida-on of a bank could be so great and the clay in such a condition of yness that when water reached the bank the whole mass might pand and slip on the foundation underneath. He thought that ose two points were worthy of consideration at the present time. e was not suggesting for one moment that modern methods of nstruction should be given up; such a suggestion would obviously futile. He did suggest, however, that it might well be dangerous adopt modern methods of construction, which were a departure om earlier practical experience, and not at the same time to adopt e methods of calculation which were produced by modern methods, d which were necessarily associated with them.

The Building Research Station had been fortunate in having some ery good friends who were members of The Institution and who had ven them facilities for investigating jobs where troubles of the pes discussed by the Author had occurred. The facilities thus ven for studying practical conditions had been of immense value. would be no exaggeration to say that the Building Research ation had learnt more from the work of the last 8 months in nnexion with failures of that character than from the whole of the ork of the previous 5 years.

It might seem that his remarks were rather far from the subject the Paper, but he would like to close by saying that many junior embers of The Institution came to see the Research Station when leave from abroad; their visits were very enjoyable, and he arned a great deal from talking to them. Almost the first question at most of them asked, however, was, "Are you doing any work soil-mechanics? Where can we obtain information about soil-echanics?" It was for that reason that he felt so strongly on the atter, appreciating the difficulties which confronted the younger en. The Institution had put its hand to the subject, through the esearch Committee and the Sub-Committee, which had been brought

[1] *Civil Engineering*, vol. 7, p. 830 (December 1937).

Dr. Stradling. into being largely by the efforts of Mr. Wentworth-Sheilds, but t work was still being hampered by lack of funds. The Governm had put up a considerable sum for the work, and as a member of t Earth Pressures Sub-Committee he could say that that Sub-Co mittee was most anxious that the work should be expanded and th its importance should be fully recognized in England.

Mr. Ellson. Mr. GEORGE ELLSON said that so much remained to be discover with regard to the characteristics of clay and its propensity to s that the present Paper was very welcome. Whilst the best metho of treating slips when they occurred should be on record, it was, in view, of greater importance to know what course to follow in ord to prevent them. The occurrence of slips in the cuttings a embankments of railways after they had once been put into use w extremely inconvenient and very costly to remedy, particula when modern fast and heavy traffic had to be maintained. Ex costs incurred during construction for the prevention of slips w small compared with the cost of repair work. Very little was rea known with regard to the angles of repose of various types of cl particularly when wet. They might vary from zero up to an ultima limit of perhaps 1 in 2, which was probably the steepest slope which any clay cutting or embankment should be made.

Clay embankments involved greater risks and danger than cl cuttings, as the results of possible slips were more serious. If, example, an embankment dropped away 6 or 8 feet below the l it was much more difficult to get the traffic going again than if t side of a cutting merely slipped on to the line. For that reason, two recent extensions of the Southern Railway, clay had not be utilized for the making of embankments except under very ri conditions. It had been preferred to import dry filling for t embankments, tipping the superfluous clay to spoil. The fill generally consisted of debris from various slum-clearance a similar schemes, and contained a great deal of hardcore. It c very little, and was extremely useful. A small amount of clay h been used in the core of the embankments, the rule laid down be that the height of the clay core should not exceed one-half the heig of the embankment from ground-level, and should in no case exce 12 feet. The remainder of the embankment, whatever its heig was tipped with dry filling superimposed over the core. As illustration of the variable character of clay, he had in mind t embankments within 20 miles of London, each 3 or 4 miles in leng and each carrying fast and heavy traffic. One of those emban ments retained its 1-in-2 side slopes fairly well, with occasional sl here and there; the other, although formed of apparently stiffer cl for the whole of its length on either side, had spread out into the m fantastic shapes and slopes. He understood from previous speak

that further research into the properties of clay was contemplated. If that resulted in giving guidance to engineers as to the safe slopes at which clay embankments and cuttings could be allowed to remain under the variable conditions of the English climate, it would be of very great importance, especially as in the next few years the construction of many new roads might be undertaken, though it remained to be seen whether any new railways would be made. The cuttings for those new roads would be subject to precisely similar conditions as railway-cuttings, but it might be that the embankments would not be subject to such punishing conditions as railway-embankments, inasmuch as road-surfacing would tend to waterproof the top of an embankment, whereas no such advantage would ordinarily be obtained by the laying of railway-tracks. Mr. Ellson.

During the last 3 years the rainfall in southern England had been generally well above the average; in 1937 it was about 41 inches. The period had been very crucial for all sorts of embankments, particularly from about December, 1936 to March, 1937. Various slips had occurred on the Southern Railway, some of which might be of interest. *Fig. 12* (facing p. 476) showed a cutting on the St. Helier branch, where slips took place on the banks that had been made about 9 years ago. The slope of the side of the cutting was 1 in 2. The lower three-fifths of the slope seemed to remain stable, but the upper part, when it was saturated with water, came down; the method of supporting that upper part by concrete arches as shown had been found to be more economical than the ordinary method of herringbone drainage. Hardcore drains were incorporated in the piers of the arches, those piers being constructed in such a manner as to allow of free percolation of water from the surrounding clay. The Author had shown cuttings with slopes of 1 in 3, but such cuttings involved much more expense for land—especially in the vicinity of London—and much greater cost in excavation. There should be some happy mean for the slope—whether 1 in 2, 1 in 2½ or 1 in 3 could probably not be said definitely without further research, and then only on the merits of each case. He mentioned that because he did not wish any one to go away with the idea that if 1-in-3 slopes were employed the problem would be solved; economic reasons might preclude such a solution.

A cutting had just been completed on the Motspur Park line with slopes of 1 in 2½. In that case perpendicular hardcore drains had been used, with herringbone drains at the top, which he considered caught all the water that came over the edge. The perpendicular legs of the drains were made in much the same way as the Author had described (pp. 465–466).

One of the most baffling slips with which he had ever had to deal had been about 2 years ago in a cutting at Botley only about 20 feet

Mr. Ellson. deep ; it had behaved in a most unreasonable manner. Every da for 8 or 9 weeks the site of the tracks was pushed up about 18 inch or 2 feet, and single-line working had to be adopted. Borings we made and it was found that the water-level was at a depth of abo 15 feet. The clay did not look particularly wet, but it kept slipping down and pushing the railway up, and nothing that w done would stop it. It was no use putting in hardcore drains in case of that kind ; the trouble was due to lateral movement of th clay, and in the end concrete stops or piers had to be put in abo 5 feet away from the side of the nearest track, and taken down in hard ground. The stops were 6 feet by 4 feet in section, and between each pair of stops steel sheet-piles were driven right throug into the hard and strutted so as to make a cut-off and prevent th movement of the clay from going under the site of the tracks. Th usual drainage-works were then carried out in the 6-foot way, a there had not been any further trouble. There was nothing ve wet or sloppy about the clay, and if any one could explain wha it was that happened he would be very grateful.

A slip had occurred at Winchfield 2 or 3 years ago at a very w period in the late autumn. The cutting at that place was abo 70 feet deep. The ground was a sandy loam which was a litt clayey, and during that rainy period it had been possible to push walking-stick right up to the handle into the soil. There was n real plane of slip, the material being very soft throughout, and became necessary to excavate about 80,000 to 90,000 cubic yards debris by mechanical diggers in order to render the bank stabl There was one particularly wet patch at the foot of the bank whe the earth was taken out to a depth of about 3 feet below rail-lev and replaced by Dungeness shingle, which was soiled over and plante with willows. (It had been found that willows absorbed a good deal water.) The finished slope of the cutting was about 1 in 2½, and th length of the slope was about 250 yards. Concrete stops were al constructed along the foot of the bank to prevent the formation the tracks being pushed up.

A slip of millions of tons of material took place in Folkeston Warren in 1915. Folkestone Warren (*Fig. 13*, facing p. 477) consiste of chalk debris overlying gault clay, the latter varying from 160 feet thickness at the back near the cliffs to a much smaller thickness the edge of the sea. The various slips of the chalk debris over th top of the gault had worn its upper surface into a rough angle leadi from the high cliffs to the foreshore ; that upper surface of the gau was apparently pitted and rugged, and held water in pools, as boring sometimes struck water, and at other times went right through int the gault without striking any water at all, even when the boring were within a few feet of one another. At one point the chalk fro

the high cliff at the back slipped down over the outer chalk detritus and went out into the sea for about a third of a mile. The islands shown in *Fig. 13* consisted of gault which was squeezed by the pressure at the foot of the high cliffs and came up on the foreshore, generally at the period of low water of spring tides, when the water-pressure on the foreshore was lowest. Mr. Ellson.

Drainage of the water from between the gault and the chalk detritus of the undercliff was effected by driving a large number of headings from the foreshore, and they released a great amount of water. For some reason during the last few years the rate of erosion of the undercliff had increased, and he had been rather concerned about that and had built a concrete wall 8 feet 6 inches thick, reinforced with old rails, on the foreshore right along the foot of the undercliff to a point where high-water mark was some distance from the railway.

The slip remained fairly quiet from 1915 until early in 1937, when there were some months of extremely heavy rainfall, between 15 and 16 inches of rain falling in the first 3 months of the year. That began to disturb the conditions. The wall had prevented the erosion of the undercliff, but resulting from the heavy rainfall another movement began at the western end of the wall. It extended over 35 acres, and perhaps half a million tons of material slipped. About a quarter of a mile of the wall was moved, the western end being pushed 90 feet out to sea. It was cracked in several places but not destroyed; in fact it did its work excellently, and held up the material behind it very well. *Fig. 14* (facing p. 477) showed how the undercliff came down. The gault in the foreground slid under the wall, which was about 18 feet high from the foundation, but did not overturn it. The groynes on the foreshore were absolutely destroyed. The wall had since been repaired and strengthened with a considerable number of buttresses.

In conclusion, he would like to refer to a cutting about 30 feet deep in very stiff clay which had recently been dealt with at Hildenborough. The clay had always been working into the ballast from below; the track was on the main boat-train route, with very heavy and fast traffic, and so about 10 years ago it had been thought advisable to put down a blanket of ashes with cross drains leading into the usual side drains. The clay, however, had continued to work up into the ballast, and after a series of wet winters it looked very unpleasant, so that drastic treatment became necessary. On consideration, it would be realized that the digging of side drains, the removal of a great deal of heavy clay, and its replacement by a blanket of ashes might be satisfactory to a certain extent, but would have the effect of removing weight and weakening the base of the cutting, thus inviting trouble. During the last 10 years that trouble

Mr. Ellson. had occurred, so that it was evidently necessary to do somethin different. He had the idea that it would be advisable, instead o using ashes, to form the blanket of some heavier material; som granite powder (waste material from a quarry) was available, whic was considerably heavier than ashes and had the same blanketin qualities. All material down to about 3 feet 6 inches below rail-lev was therefore removed. It had been in for about 10 years, and th platelayers described it to him as being rather like wet plum-pudding it had been permeated with the clay as that material worked u When the firm clay was reached a blanket of 1 foot of the grani powder was placed thereon. On the top of that paving-stones concrete 4 inches thick were laid close together, and made an excellen formation. On the top of the paving-stones, to obtain more weig to hold the formation down, four old rails were laid under each trac and four between the tracks. A layer of 3 or 4 inches of ashes w then put on the top of the paving-stones, and on the top of that th ordinary ballast was placed and the track laid. The ashes above th paving-stones would act as a cushion for the traffic, and avoid th running being too harsh. The work, which covered 1,200 yards the up and down lines, was now nearly completed.

Mr Berridge. Mr. Harold Berridge said that about 10 years ago he had bee in charge of some 40 miles of roads on clay, where there were al ten thousand houses on clay, and he had met with many of th problems discussed in the Paper. He had therefore carried out little research work himself, as the result of which he presented Paper to The Institution.[1] That Paper had subsequently bee published elsewhere.[2] Clay contained a solid constituent, whic consisted very largely of colloidal particles, the other constitue being water. The term "colloidal" had nothing mysterious abo it; it simply denoted particles too small to be visible under th microscope but larger than molecular dimensions—ranging, believed, between 10^{-4} and 10^{-8} centimetre in size. Material divid in that way had some very peculiar properties. The partic attracted each other very strongly, so that when they were saturat with water and dried out again the material did not fall to pieces a block of sand similarly treated would do; it shrank as a mass an became apparently solid, although it was porous. A 4-inch cu of surface clay which he had dug up in North London had shrun in drying to $3\frac{1}{2}$ inches, and had lost moisture equivalent to 60 gallo of water per cubic yard. In attempting to classify clays, it w

[1] Original Communication No. 4759.

[2] "The Colloidal Nature and Water-Content of Clays." *Engineering*, v 130 (1930), pp. 5, 61.

"The Physical and Mechanical Properties of Clay." *Ibid.*, vol. 132 (193 pp. 126, 192.

Mr. Berridge.

necessary to consider the ratio that Dr. Chatley, and he believed Dr. Terzaghi before him, had called ϵ, namely, the ratio of the volume of water to the volume of solid. It was quite easy to determine that ratio—in the Papers to which he had referred [1] a very simple way of doing so was given—and once that was known it was possible to appreciate the nature of the clay. From complete dryness to a mixture for which ϵ was about 0·33, the material was hard and solid and was not plastic at all in the usual sense of the word. When ϵ was between about 0·33 and about 2·7, the material was plastic, and when ϵ was more than about 2·7 it became a kind of thick and heavy liquid which could be pumped. In addition to clay having such variable properties, it was not an inert material at all. When dry clay confined in a container was wetted, it expanded with a pressure that at any rate exceeded 10 tons per square foot. That effect might evidently be important if such clay happened to be underlying a structure or the side of a bank. Further, clay contracted with considerable force when drying; he had measured contraction-pressures of up to 40 lb. per square inch. A dried briquette of the clay, of the same dimensions as a standard cement briquette, broke at a tension of 250 lb. per square inch, and the same clay required a pressure of 1,300 lb. per square inch to crush it; yet if that strong material were put into water, it would be rapidly resolved into a kind of slurry with no strength or supporting power whatever. He mentioned that to indicate how important it was that every engineer who had anything to do with clay should determine the ratio ϵ of the material with which he was dealing; that would give him a very good idea of what to expect of it.

Another point in that connexion was that the stiffness of clay increased greatly with increasing depth below ground-level. It therefore occurred to him that in forming a railway-embankment the material which came out of the bottom of a cutting ought to go into the bottom of the bank, as otherwise if the soft material from the top (which would not stand more than, say, 15 lb. per square inch) were put into the bottom of the bank, it would begin to squeeze out when the bank was built up and that pressure was exceeded. He thought that that was what very often happened.

In the construction of roads the subsoil was often disturbed by excavation for sewer-trenches, road-gully connexions, house-connexions, and water, gas and electricity services, so that there was not very much soil left undisturbed under the road. He came to the conclusion that if those trenches were taken out and the excavated clay left lying at the top of the bank, then, in the summer especially, that clay would dry to a hard and stone-like condition which was

[1] *Loc. cit.*

Mr. Berridge. very difficult to break down. If in that state it were put straight back into the trench, as was often done, it was no use trying to consolidate it. The only thing to do was to hose it and break it down until it was in a plastic condition before filling it into the trenches, and then to use mechanical ramming to consolidate it after placing. He had done that and found that it answered very well; there had been no trouble at all afterwards.

With regard to the surfacing of roads, he had found that to obtain a satisfactory result it was desirable to put down from 4 to 6 inches of clinker and to consolidate it with a fairly light (4-ton to 6-ton) roller; then when the hardcore was put on a 10-ton roller was put on it and old macadam was rolled in until there was no sign of movement at all. If wet clay squeezed up through the hardcore it had to be taken out and replaced by dry material.

Dr. Chatley. Dr. Herbert Chatley remarked that admittedly there was no entirely satisfactory definition of clay, but in view of the work that had been done by Dr. Terzaghi, which Dr. Stradling had mentioned, it would seem that the definition given in the Paper was hardly adequate. There was, moreover, an inconsistency in it, inasmuch as clay was defined both in reference to specific clay minerals and also with regard to the extreme fineness of its particles. If the term "specific clay minerals" were used, whilst it might be freely admitted that there were plastic materials which did not consist of the specific clay minerals, it could hardly be said that rocks which did not possess the specific clay minerals were clays. The fact seemed to be that the plasticity of a material depended on many things, but predominantly on the size and shape of its constituent particles, and it might or might not depend on the chemical character of the material of the particles. Four combinations of factors might therefore affect the plasticity: firstly, the size of the colloidal particles; secondly, the combination of particles of different sizes, giving different degrees of packing and spaces; thirdly, the shape of the particles, controlling the manner in which they would slide on one another; and fourthly, the chemical polarization of the surfaces of the particles, which controlled the thickness of the water films on the particles and the adhesion of the particles if the water films were removed. That made it rather difficult to define a clay at all exactly, and yet the definition and standardization of materials was essential to their understanding. He had not the faintest idea of what the various speakers who had preceded him meant by such terms as loam, clay, silt or mud, because those were vague categories which merged into one another, and he thought that they did not really mean very much until there was a standard type of each of those materials to refer to.

The Author had very strongly emphasized the importance of the

water-content of clay, and Dr. Chatley was sure that all those who had had to deal with clay would agree with him. It was Professor Terzaghi's greatest achievement that he first showed what the conditions were which controlled the water-content; until his work appeared in 1925 there was no physicist or engineer who really had any idea of the reasons for the remarkable properties of clay. Dr. Chatley.

With regard to clay slips, in some modern textbooks, especially German and Scandinavian ones, it was customary to assume a circular sliding surface for calculation—chiefly because its mathematics were very much easier than those of any other kind of surface. So far as he could judge from his own experience in Shanghai, where unfortunately he had been concerned with a large number of slips, the actual surface was generally something like a cycloid. He believed that one of the latest conclusions was that it was a logarithmic spiral, but a cycloid was much easier to work with, and it seemed to fit the shape of the surfaces better. In some of the slips that had been discussed it had been suggested that the toe of the slip actually rose. That was probably another reason why some students of the subject had favoured circular surfaces, but it would be seen from the Author's illustrations and similar figures that the surface did correspond rather well to cycloidal form.

The next question which he would like to mention was that of loading the toe of a slip. The Author had wisely drawn attention to the advantage of that method of restraining a slip in cases where it could be applied. Consideration of that method suggested that actually a continuous slope was by no means the best or most economical form for a cutting or a bank, and that a form with a berm was preferable. It saved a certain amount of excavation or filling, as the case might be, and approximated very nearly to the actual form of the slipped material after a slip; it would be very much less trouble for that form to be provided in the first place than for it to appear as the result of a slip.

His own experience was principally on slips in tidal rivers with a freeboard varying according to the tide from 4 to 5 feet up to 20 feet, and with deep water alongside. The avarice of the landowners made it necessary for the edges of the banks to be as near as possible to the water, and it had been the practice to work to a slope of 1 in 2, which in water and with the terribly bad material which existed in Shanghai involved a large element of gambling, the actual limit of stability being so nearly approached that it was a matter of considerable difficulty to get such banks to stand. Once they had stood for a week, fortunately, they usually continued to stand. No slips occurred except at the beginning of that kind of work, and as the soil consolidated and lost its water the stability became assured, unless disturbed by an earthquake.

Dr. Chatley.

The Paper was very practical and interesting, but he wished t echo Dr. Stradling's remarks with regard to the Author's omissio of any reference to some of the more recent work on the subjec Professor Terzaghi, whose name had been mentioned so often in th Discussion, might be called the father of modern soil-mechanics, and though a good mathematician, was by no means merely a theorist he had had a great deal of practical experience all over the world, s that his work could not be disregarded. Although difficult to read it was of extraordinary practical value and threw great light on th subject of the stability of clay and the time factors involved.

Mr. Carpmael.

*** Mr. RAYMOND CARPMAEL observed that remedial measure against slips on railways were divided into two classes, dealing res pectively with movements in cuttings and embankments. Th Author had dealt so efficiently with the first class that he could onl add two comments. The first was to draw attention to the dange of weakening the toe-support by the introduction of a drain (*Fig.* p. 467), which danger appeared to have been appreciated b the carrying-out of the works shown in *Fig. 6* (p. 471). Should be necessary to build a supporting wall at the toe of a slip, all portion of such a wall below levels at which water could be drained awa should be constructed of concrete or other non-pervious material. Whilst expressing his entire agreement with the Author regardin the spacing of inspection-chambers, he would emphasize th importance (laid down as an Instruction on the Great Wester Railway) of periodic examination and clearance of drains betwee the inspection-chambers. The second comment was to emphasiz the necessity of careful research into the actual causes of each sli before carrying out remedial measures. To give one instance onl in a railway-cutting about 5 chains in length, between two tunnel one slope of the cutting had remained stable for about 90 years, bu the other for over 35 years had given trouble, which for the la 15 years or so had been more or less continuous, involving th removal of about 20,000 cubic yards of material and necessitatin the purchase of valuable agricultural land. About 2 years ago larg quantities of bricks and brick-bats were found in necessary excava tions of clay, and it was ascertained from inquiries that to provide lining for the tunnels a brickyard had been opened in the cutting the cause of the slip was then apparent. The deposit of spoilt brick in one side of the cutting had created a deep water-pocket, with th result that after a long period of years the underlying clay becam softened and the slip occurred.

The treatment of slips in embankments, apart from the comm

*** This and the succeeding contributions were submitted writing.—SEC. INST. C.E.

object—provision of free passage of water—followed different lines. Mr. Carpmael.
He entirely agreed with the Author's statement in the last paragraph of p. 467. He had for some years abandoned the use of large stone or slag for bottom ballast on clay formation of new railways or widenings, and had substituted ashes.

The Author rightly drew attention to the importance of a study of the conditions of the ground on which embankments were to be tipped. The importance of the avoidance of interception of natural water-flow could hardly be overestimated, having regard also to the subsidence caused by the weight of the tipped material. He would emphasize the importance of giving special attention to the drainage of railways, and in particular to the design of structures intercepting natural watersheds. Moreover, where, as was often the case, embankments had to be of clay soil so as to give an economic balance of earthwork-quantities, special attention had to be given to bridge-substructure design, and he agreed with the Author's views on the design of wing-walls. He had had occasion when constructing a new railway to tip clay in embankments. In the endeavour to minimize obstruction to flow of water through those embankments he had tipped train-loads of ashes, about 1 of ashes to 6 of clay. Although he was aware that some engineers might regard such an action as dangerous, it appeared to have achieved its object.

To avoid the heavy expense of constructing stone or slag drains, he was investigating the results obtained by driving perforated steel pipes into the slopes either of cuttings or embankments, forcing them in by means of hydraulic jacks. So far as his investigations had at present proceeded, the elimination of water by that means, if undertaken before movement occurred, would minimize if not wholly prevent slips.

As a railway engineer, he wished that it were possible to justify from an economic standpoint the Author's ideal of a concrete blanket or mattress, having in mind the expense of carrying out the work whilst maintaining continuity of traffic. He had found that the provision of the alternative, a blanket or mattress of ashes or sand or of the two mixed, was so costly that he could only justify the expenditure on a fractional portion of his ideals. An alternative mattress of natural stone slabs (with open joints) had been tried on the Great Western Railway 30 years ago. When, as might have been expected, they, with the clay which had worked up through the joints, had to be removed, the men's task (apart from the cost of the work) was not light.

He agreed with the Author that the treatment of slips could not be governed by hard-and-fast rules. It might, however, be taken as axiomatic that all slips were due to the penetration of water, and

Mr. Carpmael. that, whatever steps were taken, the initial ones had to be such th either penetration was prevented or outlets were provided for wa that could not otherwise be diverted.

Mr. Hill. Mr. JOHN HILL observed that the Paper dealt with the practi aspects of an engineering problem, and recounted the metho adopted by the Author during several years' work on cuttings a embankments in clay soils: it was obviously not intended to be contribution to the literature on the science of soil-physics and so mechanics. It was of interest, however, to note that the simp treatment of the geological formation and physical state of clays giv by the Author on pp. 457–461 had led him to appreciate the importa influence exerted by the water-content of clay soils, and to dev practical remedial and preventative measures, which were support by recent laboratory research on the physical properties of soi Mr. L. F. Cooling, in a Paper read in 1935,[1] gave an account of so of the work on the subject carried out at the Building Resear Station, which showed that "water relationship of clay soil is prime importance as being concerned with factors which exert large influence on foundation behaviour. Particularly is this t case in connexion with shallow foundations such as exist for roads Mr. Cooling was of the opinion that, owing to the influence of moistu on the cohesive properties of clay, its shear-strength decreas considerably with increasing water-content. He also gave Te results for London clay showing a marked reduction of shear-streng when the water-content was increased beyond that normal to t clay in its undisturbed condition.

It was clear that the Author's practical measures adapted prevent extraneous water disturbing the natural cohesive streng of clay soils were supported by the conclusions arrived at as a resu of modern research on the subject. There was, however, one fact that was not specifically referred to in the Paper, namely, the possib influence of shear or compressive forces on the thixotropic properti of clay soils. In Mr. Cooling's Paper there was a reference to expe ments on the relation between maximum compressive stress an water-content of undisturbed and disturbed specimens of Lond clay. It was shown that, with specimens of such clays with simil moisture-content, manipulation of the clay was followed by a marke reduction of strength as compared with that of the undisturb specimens. That behaviour might be due to the thixotropic pr perties of clay, which, in the present connexion, might be defin as a change of consistency with the rate of application and the dur tion of shear stress. Was it possible that the changing stresses in

[1] "The Physical Properties of Clay Soils and Some Aspects of their Mechanic Behaviour." *Chemistry and Industry*, 10 Jan. 1936, p. 25.

lay foundation, produced by the passage of fast and heavy trains, Mr. Hill.
night influence the thixotropic functioning of the soil, leading to a veakening of its resistance to lateral displacement and subsequently o slips? The study of that question would call for the closest xamination of the soil, particularly as to the character of the clay-ubstance, from an aspect not covered by the Author's observations.

Mr. G. P. Manning did not suppose that any two engineers would Mr. Manning.
gree as to what constituted a typical clay. He thought that he best way to treat the problem was to imagine some perfect ypothetical substance with definite properties; each engineer ould then estimate how closely the material with which he was oncerned approximated to that perfect substance. Perfect clay, s he imagined it, was homogeneous, plastic and incompressible; hus it might be squeezed from place to place but its volume remained bsolutely constant. That might be illustrated by considering he effect of driving piles in clay. If five hundred 12-inch square biles were driven into a site 100 feet square, the volume of the biles would be 5 per cent. of the volume of the site. With the bossible exception of wet peat, no ground would absorb that volume of piles, and if the soil could not squeeze laterally into adjoining sites, the ground would rise. In general the total rise in the ground-evel was rather less than equivalent to the volume of the piles, howing that the ground was compressed and consolidated, and hat its physical properties were changed by the driving. In a perfect clay the rise in the ground would be exactly equivalent to the volume of the piles driven, meaning that the ground was squeezed from place to place but its nature was not altered. The resistance to driving in a perfect clay gradually increased as the point of the pile penetrated deeper below the surface, and depended on two factors, namely, the shear-strength of the clay, and the length of the minimum path of escape measured from the point of the pile to the ground surface. Although his own firm had never encountered an absolutely perfect clay, they had had one site where the rise in ground had been equivalent to more than 90 per cent. of the volume of the piles, with a maximum vertical rise of 2 feet 4 inches during driving and lateral displacements of setting-out pegs up to a maximum of 12 inches. It had been said that a pile bored and cast in situ would carry the same load as a pre-cast pile driven to the same depth. That was not true of most sites, but was true of a perfect clay site.

He thought that most problems connected with clay would be simplified if engineers would imagine the perfect material that he had considered and would keep in mind the two factors of shear-strength and path of escape. Most remarks in the Paper and Discussion had been directed towards increasing the shear-strength by reducing the water-content, but a foundation might be strengthened by

Mr. Manning. increasing the path of escape (for example, by loading the toe o bank with hardcore) and might be dangerously weakened by reduci the path of escape. If piles were driven through the bottom o deep narrow trench into clay and the surrounding ground w subsequently excavated down to the level of the bottom of t trench the resistance of the piles would be reduced, and might very much less than that indicated by the records of the drivi Excavation on adjoining sites also might reduce the path of esca laterally and endanger existing footings.

A remarkable example of slips in clay was afforded by the cl cliffs of the Isle of Sheppey. They were 100 feet high in plac and the shape of the slips both in elevation and plan was very clea visible.

Mr. Searle. Mr. A. B. SEARLE observed that when a chemist or a petrolog with a special knowledge of clay studied a Paper on clays writt by an engineer, the first impression received was that a differe language was being used. The engineer used terms like " clay in such different ways that repeated readings of the Paper we often necessary before it became intelligible to the chemist or t petrologist. To both the latter, the first third of the present Pap appeared to contain many unfortunate statements about cla Even the introductory definition was not free from objection, becau some clays of commercial importance were not plastic and did n develop plasticity when wetted, so that, according to the Auth they were not clays at all!

It was very doubtful whether the china-clay of Cornwall a Devon had been formed by " hot-water decomposition." Later was not usually regarded as a clay, and the term " boulder-clay had been discarded by all except a few rather old-fashioned peop The material so described varied so greatly in composition th almost any statement concerning its composition was inapplicab to some specimens. Hence, whilst some boulder-clay " contai little of true clay-substances," that statement was not true of lar masses of plastic material to which the term " boulder-clay " h been extensively applied. The Author also seemed to sugge that most sedimentary clays were of fluviatile origin, whereas tho of lacustrine and marine origin were far more extensive—at a rate in the British Isles. What he appeared to mean was that " the London area " the sedimentary clays were mainly riv transported clays which had been deposited in lakes or estuaries.

The Author also used the term " clay " in a very restricted sen to mean compacted mud or partially-dried mud, namely, mater intermediate between mud and shale. That use of the term had be borne in mind when considering his Paper, because on some pag he used it with a different meaning. When the Author referred to

clay structure " he seemed to mean that of a stiff paste, or what a brick-maker might call a " stiff plastic mass " ; he was not referring to the arrangement of the atoms to form a unit cell of clay. The statement that " the term ' clay ' as the civil engineer knows it is applied generally to brown clay " was only correct with regard to certain areas : it was not of general application. What the Author appeared to mean was that in " the London area " the upper strata were composed of brown clay and the lower ones of blue clay, and that the latter were more stable than the former from an engineering point of view. Mr. Searle.

When the Author reached the engineering part of his Paper (at the foot of p. 461) his statements were much more accurate and the information he imparted was very useful. Indeed, railway- and other engineers should be grateful to him for setting down so lucidly some of the results of his experience. In making roads or railways on clay land, it should never be forgotten that such clay was mobile and not truly solid. It yielded under pressure like a highly-viscous liquid, and the first essential was to spread the pressure over as large an area as possible. For that purpose a properly-designed raft or mattress of reinforced concrete was the nearest approach to the ideal, and the Author rightly advocated it. Where such a raft was too costly, the best substitute was a bed of ashes overlain by a thinner layer of crushed stone or slag, as the Author suggested.

The Author rightly said that clay slides[1] were often caused by the clay drying and then being wetted by rain. The best remedy would be to prevent the clay from drying, but in most cases that was impracticable, and so the cracks resulting from drying had to be filled as they occurred. Any suitable covering would probably lessen the drying and so reduce the risk of clay slides; but even the cheapest covering was expensive. Thin sheets of concrete would probably be the most effective cover; next in efficacy was a thick porous layer which would prevent drying in hot weather and would absorb and drain away rain during wet weather. There again, the Author's experience had led him to make sound recommendations. His emphasis on the importance of draining as well as of supporting the material which tended to slide was very sound, and would be heartily endorsed by all engineers who had had much experience of the problem.

The Author wisely condemned what was known as " burning " as a method of preventing clay slides. If a great depth of material could be excavated, dried, ground to powder, about half of it burned

[1] The term *clay slip* was better avoided, as it had another meaning in technology.

Mr. Searle. properly (as bricks were burned) and the burned product ground powder, thoroughly mixed with the raw material, and the who replaced, rammed, and covered to make it rain-proof, satisfacto results might be obtained, but at a prohibitive cost. As nothi short of that could be relied upon, the customary method "burning" was uncertain and should, in all important situation be avoided.

It might, with advantage, have been explained more fully in t Paper that the mobility of clay was due to layers of water pen trating between the sheets of aluminium and silicon ions formi the unit cell of the clay-molecule and also to the water adhering the surface of the minute particles of clay. By thus separating t particles and providing a well-lubricated surface on which th might slide, a very mobile, semi-fluid mass was produced, who weight was sufficient (without any external pressure) to cause it slide. The drier the mass the greater was its resistance to mov ment, but as soon as sufficient water had penetrated—particular at a low level—the more favourable were the conditions for a slid That was the reason why so many clay-slides were caused by wat accumulated in the mass, particularly in cuttings and embankment Adequate covering to keep the mass dry, as well as adequate draina to remove any water accidentally entering it, were essential to t prevention of clay-slides. The Author did not definitely menti multiple drainage at several levels, though that was advisable important situations. Drain-pipes made of porous material ha several obvious advantages over the ordinary drain-pipes : they we extensively used on the Continent.

The Author's descriptions of several slides and of the mea adopted to deal with them showed how clearly he had realized t four first principles in dealing with land-slides ; namely :—

(*a*) To drain the material throughout, but especially at t lower level. That might have to be at a considerab depth below rail- or road-level if it were to be effective.

(*b*) To support the toe of the possibly-moving mass so as t restrict its movement as much as possible.

(*c*) To break up the mass into a series of small ones, and tre each separately by methods (*a*) and (*b*).

(*d*) To avoid excessively steep slopes, because the angle of repo of wet clay was very small, sometimes as low as 1 degrees.

The Author had performed a very useful service in publishi his Paper, which should be of great practical value to many enginee

Mr. Walker. Mr. E. G. WALKER observed that, whilst the practical problem met in dealing with clay soils had been appreciated from the earlie

Mr. Walker.

days of engineering, it was remarkable that only in comparatively recent times had any appreciable advance been made in the acquisition of the knowledge necessary to enable the incidence of slips to be minimized. The Author had given a useful account of methods which had been employed successfully in an area where natural conditions were bad. In general terms, his methods all depended upon one cardinal principle, namely, the provision of adequate permanent drainage, so that neither could excess of water accumulate at any point at which the stability of the slopes could be affected by it, nor could deficiency of moisture render the slopes pervious. Unfortunately, the natural range of variation of water-content was greater than the range over which the stability of the natural slope could be maintained. Hence the variations of the physical characteristics of the clay—its cohesion, compressive strength, shearing strength, etc.—with varying moisture-content had to be taken into account in a way that they had not been in the past. Those matters were not touched upon in the Paper, a fact which emphasized the necessity for increased effort to enlarge the scope of the work on the mechanics of soil which was being done at the Building Research Station with the co-operation of the Institution's Research Sub-Committee on Earth Pressures. Those studies had developed practical methods of studying earth slopes, taking into consideration the physical properties of the various strata forming and underlying the slope. They had been applied to the investigation of slips after they had occurred and had enabled the causes of failure to be ascertained. It was of great importance that that work should be extended and made available. The Author on p. 464 referred to an expenditure of the order of £260 per annum per mile in one railway district. Probably that was considerably greater than the average for the whole country, but the aggregate annual cost was evidently great. If to that were added the cost of repairing slips on reservoir-banks, roads, and the numerous other earth cuttings and banks that were to be found, the aggregate annual cost of earth slips in Great Britain might well reach a sum of the order of half-a-million pounds, apart from the large accident-risks. It did not seem too much, therefore, to ask that the large public and semi-public authorities who were responsible for the maintenance of those works should give substantial financial support to enable the scale of the work of the Earth Pressures Sub-Committee—at present strictly limited by the funds at its disposal—to be increased so as to enable practical solutions to be found to the engineering problems of the control of earth structures (including foundations).

The Author.

The AUTHOR, in reply, observed that criticism of the Paper had been confined to the Introduction, and had been made mainly by soil-physicists or ceramic specialists. In considering the Intro-

The Author. duction, it should be borne in mind that it was in no sense a revi of the research-work that had been carried out in connexion wi clay soils, any one branch of which might form the subject of Paper. Its object was to describe briefly clays—in particular, se mentary clays—and to trace their origins. It gave, he suggested clear description of the material from the point of view of t engineer who had had to solve practical problems connected wi clay soils; in fact, the Author had definitely based his methods the information contained in it. He was aware of some of t research work that had been carried out, and submitted that t confirmed the conclusions he had reached by consideration of t subject as outlined in the Introduction. Much research work of practical character remained to be done in connexion with clay soi and the Author's object would have been achieved if his Pa resulted in another Paper being submitted dealing with the research into the subject and their application to practical problems.

Dr. Stradling's references to modern methods of handling ear were interesting. It was, however, regrettable that he had not be more specific as to the particular methods to which he referr The engineer had usually to consider the economic factor in works. If, however, he could be given a definite lead by t scientist as to the best technique to adopt to ensure the stability soils, he could in many instances construct his earthworks accor ingly.

Dr. Stradling hardly flattered the value of the research work th had been done in Great Britain when he stated that the Buildi Research Station had learnt more from investigating practi problems during the last 8 months than from the whole of the wo —presumably the laboratory research on soil-mechanics—of t previous 5 years. Such a statement almost justified the omissi by the Author in his Introduction to refer to the subject of so mechanics, about which Dr. Stradling felt so strongly.

The remarks made by Mr. Ellson were most informative, and would be particularly interesting to note the behaviour under tra of the paved road-bed he described.

The suggestion of Mr. Berridge regarding the formation of emba ments was usually followed as far as was possible. If, however cutting was of appreciable depth it had necessarily to be excavat in lifts, starting from the top. In new works at present bei carried out under the Author's supervision it had been fou economically possible, owing to the magnitude, character and lo tion of the several works, to form the embankments of select material and to run unsatisfactory clay soils to spoil. There w no doubt that the modern drag-line excavator was the best tool use in clay soils when they had to be used to form embankments,

that it broke up the material, which subsequently consolidated far better than if a digger had been used, especially if the latter had a high-capacity bucket. The Author.

The Author was particularly interested in Dr. Chatley's suggestion that a varying inclination might be advantageous for a cutting or embankment; he had also considered the point, but had found that in clay, apart from the greater liability of slips, if a steeper slope than 1 in 3 were used the surfacing soil or ashes was often washed down, unless the slopes were turfed, which was usually a practical impossibility. Varying inclinations of slopes in cuttings were, of course, often used if they were through different classes of soils.

Mr. Carpmael had drawn attention to a most important point with regard to the drain at the toe of a clay cutting. The Author's experience was that such drains should always be placed on a substantial bed of concrete, the trench being well and carefully packed so as not to weaken the toe; in bad ground he would prefer also to reinforce the concrete somewhat as shown in *Fig. 6*, p. 471. Mr. Carpmael also mentioned a point on which there was a difference of opinion among engineers, namely, the incorporation of ashes with clay in embankments. It would no doubt be of interest to Mr. Carpmael that the Author had had the same success; he, however, considered it advisable that the ashes should be well incorporated with the clay, and would prefer to have them in layers, sloping outwards.

From the point of view of the technologist the contribution of Mr. Hill appeared to be one of the most constructive; moreover, he saw the Introduction of the Paper in its intended perspective. His remarks suggested a most interesting line of research, which the Author would commend to his scientific colleagues on the Research Sub-Committee on Earth Pressures.

The observations of Mr. Manning indicated an interesting line of research in connexion with piling. The clay cliffs of Sheppey were similar to those at Frinton.

Mr. Searle criticized the Introduction on the ground that the Author had not used the language of the chemist or petrologist. However, the Paper dealt with engineering problems associated with clay, and aimed to be of practical use to engineers, so that the Author might be pardoned if he chose to use the language of the engineer.

In a strict attempt to classify clays it would be possible to refer to clays that did not exhibit marked plasticity, but in the literature on the subject the possession of some degree of plasticity was inevitably stressed as a dominant feature of all but a few clays. Descriptions of argillaceous rocks were still in most cases arbitrary, and biassed according to the particular angle from which the writer

The Author. viewed the subject, as witnessed by the classifications enunciated b several authorities, including Mr. Searle. Mr. Searle himself regarde clay from the viewpoint of a ceramic technologist, for in his artic on Clay in the " Encyclopædia of Ceramic Industries " he considere it " reasonable to define clay as a mineral consisting essentially an alumino-silicic acid in such physical state that when mixed wit water it produces a plastic paste. On this definition brick-clays fire-clays, etc., would be rightly regarded as very impure clay containing in addition to the essential clay-substance such minera as quartz, felspar, and mica, etc." That definition from the ceram point of view ignored the finely-divided nature of the foreign matte which, the Author contended, had great effect on the slip properti of clay. With regard to Mr. Searle's reference to the use of the ter " boulder clay," in the same article it was stated :—" While geol gists use the term ' boulder clay ' to include the whole of the [glaci drift] deposit, brick manufacturers and others more usually unde stand it to be confined to the sticky plastic portions of the who material "—again the ceramic viewpoint.

The Author ventured to think that his own conception of exce water acting as a lubricant, and permitting the particles to slip ov each other when a stress was applied to the clay, was as helpful t the practical engineer as the more academic wording, suggested b Mr. Searle, that the mobility of clay was due to layers of wat penetrating between the sheets of aluminium and silicon ions formin the unit cell of the clay-molecule. The two conceptions were essen tially similar, though on different scales, Mr. Searle having sough additional aid from the technique of the X-ray diffraction camera.

The Author noted with appreciation Mr. Searle's confirmation o his practical methods.

*** The Correspondence on the foregoing Paper will be publishe in the Institution Journal for October, 1938.—SEC. INST. C.E.

JOINT MEETING.

1 March, 1938.

FREDERICK CHARLES COOK, C.B., D.S.O., M.C., Chief Engineer, Ministry of Transport, in the Chair.

A Joint Meeting, organized by the Institution of Automobile Engineers, was held in the Great Hall of the Institution of Civil Engineers with :—

The Association of Public Lighting Engineers,
The Institution of Automobile Engineers,
The Institution of Electrical Engineers,
The Institution of Engineers-in-Charge,
The Institution of Highway Engineers,
The Institution of Gas Engineers,
The Institution of Municipal and County Engineers,
The Institution of Petroleum Technologists,
The Institute of Fuel,
The Institute of Marine Engineers,
The Institute of Transport,
The Iron and Steel Institute,
The Junior Institution of Engineers,
The National Safety First Association,
The Roads Improvements Association,
The Town Planning Institute,

at which three Papers on "Essential Road Conditions Governing the Safety of Modern Traffic" [1] were presented.

Introductory Remarks.

Major-General S. C. Peck stated that he had been asked, as President of the Institution of Automobile Engineers, to make an introductory statement, indicating some of the unsatisfactory road conditions existing in Great Britain, and so forming a background for the Papers that were being presented. The appalling number of road accidents was without doubt the most important question before

[1] Journal I.A.E., vol. vi, No. 6, p. 14 (March, 1938). Complete with discussion and written contributions in Proc. I.A.E., vol. xxxii, to be issued about September, 1938.

the public, not only of Great Britain but of every other country in the world to-day. Some countries had succeeded in tackling that problem better than others, but he thought it would be agreed that generally speaking they might all be said to have failed in view of the large number of accidents in every country. In 1937 alone nearly 30,000 people were involved in accidents on the roads of Great Britain, approximately 3 per cent. of which were fatal.

At the Public Works Congress Dinner in London on November 17, 1937, the Minister of Transport had stated that only 3 in every 200 road accidents in Great Britain were due to road defects, but he felt it very difficult to accept that statement. He dared to suggest that most accidents, other than those caused by inexperienced drivers, were caused by one or more of the following features of British roads, and that those were capable, at any rate, to a certain extent, of being obviated :—lack of uniform width ; lack of separate lanes for one-way traffic on main roads and of special provision for cyclists ; bad road-junctions ; inefficient and varying illumination of roads ; the unrestricted development of housing estates, factory sites, advertisements and signs along the main highways ; lack of non-skid road surfaces ; and badly planned curves, gradients and banking.

He submitted that the present-day state of affairs was due to the fact that those responsible for the roads had failed to foresee the very rapid increase in the number of road vehicles which had taken place during the last 20 years. He would remind them that in 1917 there were about 188,000 vehicles registered, whereas 20 years later, in 1937, there were about 2,930,000 registrations. During that period, on the other hand, the safety of the motor vehicle itself had steadily increased and that had tended to lower the casualty figures. Safety had been increased by the great improvements which had been effected in the braking systems, in reliability (freedom from mechanical breakdown), in easy gear-changes (which had helped to eliminate fatigue in drivers), and in increased acceleration, and by the provision of small matters such as wind-screen wipers, dipping lamps, improved tires and improved steering. Modern vehicles were faster than earlier vehicles, but they were also more inherently safe to drive, and therefore they were safer from the point of view of the pedestrian than were the vehicles of several years ago. It was hoped that the three Papers presented would focus attention upon that most important matter of road safety, and would show how the road planning of Great Britain could really be improved. The resources of the Institutions taking part in the Joint Meeting covered a wide field, and, if made use of, could not fail to render a good account of themselves in helping the Ministry in its very difficult task of lowering the casualty lists.

SECTION I.—ROAD PLANNING.

By THOMAS ADAMS, D.Eng., F.R.I.B.A.

(*Abridged.*)

FUNCTIONS OF ROADS AND THEIR RELATION TO COMMERCE AND PUBLIC WELFARE IN GREAT BRITAIN.

Primarily, the planning and layout of roads,[1] as of other ways of communication, is a problem in economics and only secondarily a problem in design and engineering. Before any sound conception of policy or method in planning roads can be arrived at, a true understanding of the economic principles underlying their use in society must be obtained. The true position is that a properly conceived and intelligently planned road system is the most universally beneficial of all public works. Only when this position is appreciated by the public, and those who control the public purse, will the wisdom be perceived of investing in road improvements all revenues derived from vehicles using the roads and, in addition, such revenues as can reasonably and practicably be collected from land developers and commercial interests that derive benefit from roads.

Therefore the national interest in securing an adequate and well-planned system of roads is not confined to considerations of safety or to lessening costs and inconveniences of traffic congestion, however important these are as separate matters, but to considerations that are vital to general welfare in its broadest implications.

The immediate problem is—What constitutes an adequate and properly planned system of roads for general welfare in relation to existing facts and tendencies? and therein lies the opening for differences of opinion. The problem must be faced with foresight and prudence, and without extravagance.

In democratic countries such as in Great Britain the Government is inclined to be prudent to the point of short-sightedness, whereas in more despotic countries the virtue of foresight often leads to extravagance. Whilst Britain may envy Germany's present road construction plans, no underestimate should be made of the values of the traditional British methods of proceeding by gradual evolution in meeting new demands, of maintaining the strength of local

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[1] In this Paper the word "road" is used in the generic sense as comprehending a highway, street, road or road reservation; where the word "highway" is used, however, it is intended to have the meaning of a public road on which all have a right to go, and where a "street" is used it means a road in a town, lined with buildings

governments in face of tendencies towards centralization, and of flexible rather than arbitrary methods of administration and execution.

Moreover, it should be borne in mind that, however backward Great Britain may be regarding the building of new trunk roads for motor vehicles, it possesses a network of tributary roads as a nucleus for expansion to meet modern needs which is superior in the quality of its surfaces and in general utility to that of any country. Great Britain is now, however, confronted with the necessity of adopting a bolder policy in road planning than hitherto—if the full benefits of motor transport are to be secured and its toll of human life lessened. The Author would emphasize that there is no need to imitate the methods of any other country having different economic and political conditions and a different conception of what constitutes social welfare.

In its modern aspects, the task of road improvement for general welfare in Great Britain consists of developing a national system of new express roads as an addition to the system of existing tributary roads, and of maintaining a proper balance and reasonable fluidity between the two, or, in other words, to plan new roads and train existing highways and streets together so as to form a national pattern.

National Planning of Roads.

The Author stated that proposals for national planning of roads had been before the public of England for 30 years, and after referring to the passing of the Town Planning Act and the creation of the Road Board, and also to the Arterial Roads Conference in Greater London, set up by Mr. John Burns in 1913, and to the International Road Congress which met in London that year, he said that his suggestions for planning a national road system had to be confined to stating a few broad considerations regarding fundamental aspects of the problem.

Fundamental Aspects in Road Design.

The primary aim in planning roads in the technical sense is to reduce the costs of overcoming the friction of space that needs to be traversed between any two points. These costs may be defined in simple terms as distance-cost and time-cost, remembering that the latter is more important than the former, especially in the

crowded roads of large towns. But there must be included costs in terms of human energy—such as physical and nervous strain, danger to life and limb, inconvenience and discomfort; and of mechanical energy, such as wear and tear of vehicles to overcome obstructions as well as distances; and, in addition, under modern conditions, depreciation of property values by reason of interference with the economic uses of buildings and land on frontages of roads in the interest of traffic.

The reduction of these complicated costs involves consideration of many more problems than directness of route and ease of gradients. Mileage costs for motor vehicles in traversing large towns include many factors that are incalculable, which is one of the excuses for allowing them to mount to almost intolerable proportions.

Broad considerations in regard to methods of design of a road system for national development include:

(1) Planning the national system of trunk roads, including new trunk lines, as the backbone of the system in its ultimate pattern, in relation to and in co-ordination with all existing highways, waterways, railways, and airports. (The inclusion of existing highways in a new road system should be subordinate to the new system instead of the reverse.)

(2) Arrangement of lines of new express roads avoiding sites that are in close proximity to existing highways with their numerous branch connexions and high values of frontage land.

(3) Design of alignment of all new roads and re-alignment of existing roads, so as to secure steady flow of traffic and reduce distances between points of origin and destination of traffic, compatible with ease of gradient and avoidance of natural and artificial obstructions to steady flow.

(4) Adaptability of widths, designs of cross-sections and surfaces of roads to varied uses, and to the different needs of traffic and requirements of safety instead of conformity throughout to a standardized system.

(5) Design of junctions so as to facilitate free movement and safe conditions for crossing traffic, while avoiding unnecessary high cost or waste of space.

(6) Segregation of different types of vehicles or road users into different roads or paths, including the making of special provision for pedestrians, cyclists, and riders, where necessary, for obtaining the steady flow of traffic and conditions of safety.

(7) Maintenance of flexibility in design of trunk and express roads so as to permit safe and steady interweaving of traffic from the branch system of highways.

(8) Fitting of alignments and widths of highways and streets to areas built upon or having a potentiality for building development, and conversely, fitting of buildings to the highway or street system in built areas.

(9) Generally, the promotion of a system of a safe and steady traffic flow at reasonable speeds for all road traffic in town and country.

It is obvious that the greatest difficulties in giving effect to these ideals is in connexion with the approaches to and intersections of towns. It will be largely waste of effort to plan a national system of highways in England which brings traffic in greater volume to the periphery of large centres and fails to provide facilities for improvement of communication within and through them.

The principles set forth in this Paper do not conflict with those in the Memorandum on the " Lay-out and Construction of Roads " issued by the Ministry of Transport.[1] The importance stressed on the need of an investigation of traffic conditions dealing with a definite area as a preliminary to planning (Section 2) is an argument also for national or large regional surveys. One of the greatest needs is for an aerial map of the whole country, which might be made by the Air Ministry in training its staff, and would add enormously to the utility of the Ordnance Survey maps.

On the controversial question of what is the major cause of accidents (Section 3), the Author ventures two opinions. First, that a great many accidents that are attributable to the actions of persons would, in spite of these actions, be unlikely to have occurred but for some imperfection in the plan of the highway, and, secondly, that the fact that so many accidents occur on improved roads is largely due to the underlying fact that the surfaces of these roads have been made perfect for speeding without the roads having been properly replanned to suit the greatly improved conditions of the surface.

In America and in Germany they have found out that roadside " beautification " is a palliative for relieving the effects of bad design, and that every new or widened road should be designed jointly by the engineer and a trained landscape architect, so that as a whole it will become an amenity, and not an ugly scar made better or worse in appearance by attempted " beautification."

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[1] Ministry of Transport, Memorandum on the Lay-out and Construction of Roads. No. 483 (Roads). H.M. Stationery Office, 1937.

Bridges, as an integral part of roads, may be the most pleasing feature in their architectural design, and the pleasure they give will be in the degrees to which their design reveals qualities of simplicity, conformity to purpose, absence of sham effects, suitability of materials, fitness to the topography, and regard for principles of scale and proportion.

Prevention of confusing and ugly signs and notice boards, of obtrusive and sometimes dangerous advertising hoardings, and elimination of many obsolete signs, are, however, necessary for improving appearance as well as utility of roads.

The Ministry's Memorandum does not raise many controversial points. There is common agreement regarding unit widths; utility of dual carriageways; amplitude of margins, central reservations, and space for junctions and roundabouts; application of super-elevation; providing adequate visibility at vertical and horizontal curves, and for separation of traffic by lines, etc. There is, however, evidence of the need of better practice in complying with the Ministry's suggestion that highways and islands should not be obstructed by erections, as they are on such important roads as the Kingston By-pass.

The Minister also drew attention to a matter affecting planning, namely, the fact that commercial vehicles included 6-wheelers 30 feet long. It is not unreasonable to raise the question whether Great Britain can afford to build a system of roads to accommodate, at fast speeds and under safe conditions, commercial vehicles of unlimited size and with an unrestricted number of trailers. If segregation becomes compulsory by reason of defective roads, it is likely to lead to difficulties and controversy. On the other hand, it is important that a policy in regard to segregation should be determined in advance of designing and building a new road system. On new motor roads, however, the segregation should be in the form of dividing fast from slow traffic rather than of separating different types of vehicular traffic.

Steps Toward a Solution of Road-Planning Problems.

The first and most urgent step towards a solution is to promote research into problems of planning as fully and intensively as has been carried out with regard to problems of construction. This research should be conducted on a nation-wide basis with three main objects—firstly, to obtain essential facts based on a survey; secondly, to make a plan of appropriate sites for new road reservations between and surrounding towns, followed by the acquisition

of sites; and thirdly, to elaborate more fully than is now being done definite road proposals for improving communications through towns.

The three objects may be briefly described as follows:—

(1) *Survey.* What is required is to obtain a fuller and more comprehensive understanding than is now available of the extent and degree to which improvements can be made in the design of existing roads in all their aspects as traffic ways; but also, and simultaneously, to discover their relationship to building development, to landscape features, and to anticipated possibilities connected with the building of entirely new roads.

Such a study is necessary to secure the knowledge required to make a positive plan of road improvements, and to get rid of excessively hampering restrictions of traffic in the alleged interest of safety and building on the one hand, and of building development in the alleged interest of safety and traffic on the other.

(2) *Plan of Reservations for New Roads.* On the assumption that sufficient information is already available to prove the necessity of building a system of roads for motor traffic on new sites in the near future, the second object is to obtain a plan showing the most appropriate reservations of land that are available in Great Britain, on which new routes can be established, expecting that the preparation of this plan will be followed by the acquisition of sites.

Until a plan showing possible reservations of land is available, the building of new roads must be haphazard and cannot be economically carried out. The following are among the considerations that should be borne in mind in making the plan of reservations:

(*a*) The aim should be to reserve a site of not less than 160 feet wide to permit triple roads, two for fast traffic moving in opposite directions and one for slow traffic with adequate provision for margins, footpaths, etc., as hereinafter described. Building development should be permitted on one side only, namely, that which fronts on the slow traffic road, which would replace all service roads.

(*b*) Close contiguity to lines of existing "main" roads should be avoided and lines reserved close to railways, canals, rivers, etc., partly to make it unnecessary to compensate for severance. Topographical advantages are usually greatest along such lines.

(*c*) Cross roads should be planned over or under the new road reservation, where this is facilitated by topographical conditions. In level areas, adequate roundabouts in the traffic routes should be sufficient.

(*d*) Where the road reservation must intersect beautiful country and form approaches to parks or residential areas, the reservation should be planned in the form of a parkway, with wider margins and central strips and entire prevention of ribbon development on both sides.

(*e*) As a rule the triple roads reservation should not enter a town, but connect with a similar reservation made surrounding the town.

(*f*) The proposed triple road reservation is based on acceptance of the fact that in Great Britain there must be a high degree of fluidity of traffic passing between new and existing roads, and between fast and slow routes in the interest of commerce and economic land development. It also assumes the necessity of group and ribbon development in close contiguity to the main road system and provides for this on one side only without any frontage on or access to the fast-traffic routes. The present method of restricting ribbon development is injurious to land development, without effecting more than a widening of the ribbons and therefore any real improvement in traffic conditions or amenities. A further assumption is that fast-traffic routes should not have more than two lanes moving in one direction. The slow-traffic route would provide partial relief to the fast-traffic routes during hours of peak, and this is better, having regard to other advantages, than having dual carriageways of three lanes, each without the parallel slow-traffic route.

(3) *Road* (*Street*) *Planning in Towns.* The third object is that bolder and more constructive methods of planning the interior arteries of towns must be promoted to link them up with the national system by new circular roads on the periphery of towns.

The three most fundamental principles are—that street planning and building development must be dealt with as one problem ; that more adequate provision should be made for circular flow of traffic ; and that in dealing with existing streets each should be considered as a distinct problem of design, as well as part of a comprehensive plan.

In regard to general traffic circulation, the Author agrees with the report of The Royal Automobile Club on London Traffic that perhaps the greatest need in London is more adequate provision for circular flow of traffic.[1] Another urgent problem of road planning in London

[1] Memorandum on London Traffic, R.A.C. December 8th, 1937.

is to provide for parking space for cars off the street, including underground spaces similar to those constructed in Hastings.[1]

Conclusion.

Dealing, as this Paper does, with the large and complicated problem of road planning independently of matters relating to construction and details of design, it has not been possible to touch all aspects relating to design. The Author hopes, however, that he has given a broad picture of some aspects of special importance and has thereby raised issues that will stimulate discussion and be provocative of thoughtful attention on the part of those who have it in their power to give direction to future policies and methods in the planning of roads.

Section II.—ROAD CONSTRUCTION.

By Cecil Lee Howard Humphreys, T.D., M. Inst. C.E., M.I. Mech. E.

(*Abridged.*)

Introduction.

This Paper is intended specifically to deal with the details of the design and, consequently, of the construction of roads as regards safety. It is not possible to put forward any revolutionary changes in design. Such things are evolved slowly, partly from experience and partly from research.

Tendencies of Vehicle Design and Road Development.

Future road design cannot be considered without first envisaging the probable tendencies of vehicles. Recent inquiries on a large scale in America [2] suggest that the average driver does not want excessive speed, but does want to drive at 60 miles per hour without effort. It is also thought that any future passenger car will be able to negotiate a 10-per-cent. gradient without effort. The limit has

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[1] Sidney Little, " A New Sea Wall, Promenade and Underground Parking Station, etc., Hastings." Journal Inst. M. & Cy. E., vol. lviii (1931), p. 593.

[2] *Civil Engineering.* Annual Convention Paper. Vol. 7 (1937), p. 683 (October, 1937).

been reached, it is believed, in tire design from the point of view of road-holding qualities, and a co-efficient of friction of 0·6 between tire and road is regarded as a maximum. Sizes of cars will not alter much, although doors will remain low and will consequently affect the height of curbs. Road clearance will not be reduced below 8 inches. There is talk of the separation, in some cases, of goods from passenger traffic, and there is a demand for high-speed highways.

These views are not at all unfitted to be held in Great Britain, though it is questionable whether a 10-per-cent. gradient is desirable, except in cases of great engineering difficulty. There is a tendency in Great Britain for cars to become more powerful, for their build to be low, and for the motorist to wish to drive in comfort and without strain at a reasonable speed. In France the need for motor roads is not so great, for outside of the great towns the traffic density is relatively small, and France has long had her "Routes Nationales." By-passes around her cities will therefore serve her ends for some time to come. Motor roads, "express highways" as they are called in the United States of America, *autobahnen* in Germany, or *autostrade* in Italy, have, in the opinion of the Author, come to stay.

The motor road as laid out in Germany [1] has no pedestrians, no cyclists, no advertisements, no cross roads, no buildings, and access to it for vehicles is at intervals of from 9 to 12 miles. In the result it is readily possible to keep up a high average speed without exceeding 50 to 55 miles per hour and without fatigue.

The very words "high-speed motor road" conjure up all forms of objection, of prejudice, and of loose thinking, but until one has been built—why not one from London to Brighton or Dover?—and given a fair trial, there will be no means of forming an equitable judgment upon this matter, for, as is so often said, foreign conditions are not English conditions. Used at the speed at which the modern touring car will cruise, such roads should be a blessing to the country, and should tend, if rightly designed, to cause the driver to do the safe thing. The Author wishes to emphasize that it is the sense of irritation and frustration engendered by the overcrowding of our roads which causes so many a senseless risk to be taken.

There are other objections which are commonly put forward. That motor roads are ugly—this is entirely untrue if German roads be taken as a model. It is said that England is too small for such roads; this is again a matter of degree. The width of road required for an American express highway is estimated at 300 feet; an *autobahn* can be accommodated in 100 feet; the greatest width normally

[1] Preliminary Report by the Members of the Parliamentary Road Group on the German Roads Delegation on the subject of their visit to Germany, p. 8.

contemplated for an English arterial road is 140 feet, so that the necessary acquisition of land is already officially envisaged without a qualm. Another objection is that the cost is alleged to be impossible. Possibly the true cost is about £80,000 per mile, and a skeleton system of 2,000 miles would thus cost £160,000,000.

Details of Design for Roads in General.

At this stage it is desirable to draw the attention of the lay reader to the "Memorandum on the Layout and Construction of Roads," No. 483, published by the Minister of Transport in January, 1937,[1] to supersede Memorandum No. 336 of August, 1930. This booklet contains, without an excess of theory, the consolidated experience of the Department in the light of the policy which has so far been followed.

(1) *Speeds.*

It may be considered to be a part of the settled policy of Great Britain that in built-up areas the 30 miles per hour speed limit shall remain permanently. Outside these areas the Author considers that speeds up to 80 miles per hour should be taken as the basis of design. If a steady average of 50 miles per hour can be kept up on the best roads it is enough for the dimensions of this small island.

(2) *Widths.*

It is recommended in British practice that the width of a road with cycle tracks, etc., should be up to 140 feet from fence to fence to allow for improved amenities. In America parkways are up to 200 feet wide, and for express highways even 500 feet has been mentioned. (As perhaps some guide to the magnitude of the problem it may be mentioned that a 100-foot strip of land represents 12 acres per mile of road. Obviously if the land is agricultural the extra cost of land is negligible in comparison with the other costs.)

By the time that cycle paths and footpaths have been provided the 140-foot width leaves little margin for beautifying the roads, and it is perhaps worth while to consider whether, in spite of the congestion of this island of Great Britain, a more spacious layout should not be encouraged.

As to the width of each traffic lane, there is a great deal of comfort and safety to be derived from a lane of ample width, and the Author is of opinion that the German dimensions of about 13 feet per lane

[1] Footnote (1), p. 507.

make for extremely pleasant and safe fast running. Certainly a 10-foot lane is too narrow for comfort.

Where there are dual carriageways (namely, for traffic exceeding 400 vehicles per hour) the width of the centre strip is of great importance. It should be wide enough to allow a vehicle to wait transversely to the lines of traffic while it is turning from the " up " to the " down " road or vice versa without having either its front or its rear protruding into a carriageway, and, if this is done and the central strip is planted with a hedge, it will be found that the headlight glare problem is to a large extent solved by the separation and cover so provided.

(3) *Gradients.*

With self-propelled traffic these are a somewhat minor factor, certainly as regards safety. It is of interest to note that practice here suggests that heavy traffic tends to avoid routes graded much in excess of 3 per cent.,[1] but in America gradients of 5 per cent. are considered [2] to be the limit for express roads. German practice varies between 4 and 7 per cent. on the *autobahnen*, while Italy prefers 3 per cent. on the *autostrade*. Probably 5 per cent. is a good limit for new road construction.

Gradients should be as long and steady as possible, because on a straight road a switchback profile renders night driving unduly troublesome, owing to the effect of the headlights of vehicles.

(4) *Horizontal Curvature.*

Curvature must clearly be governed, as regards safety, by the distance in which a car can stop. English practice adopts 1,000 feet as a minimum radius. America suggests as much as 3,000 feet as a minimum for express highways, and German practice on *autobahnen* is from 5,900 feet to 1,960 feet, according to the class of the road. It is invariably desirable to use transition curves.

(5) *Vertical Curvature.*

Practice in Great Britain recommends a visibility of 500 feet. Similar principles of design are accepted abroad.

(6) *Super-elevation and Camber.*

A clear limit to super-elevation exists if the road is to take mixed

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[1] F. C. Cook, " Road Design and Road Safety." Journal Inst. C.E., vol. 4 (1936–37), p. 177. (December 1936.)

[2] C. M. Noble, " The Modern Express Highway." Proc. Am. Soc. C.E., vol. 63 (1937, Part 8, No. 2), p. 1074.

traffic, and, in American and German practice, this is considered to be from 8 to 10 per cent. as against the British 6 per cent. Slow vehicles and horses must be able to negotiate roads without sliding sideways down their surfaces.

The question of super-elevation could well be combined with curvature and surface friction for purposes of experimental study.

It is difficult to see where the need now exists for the use of camber on roads. It is necessary to throw off the water at a rate sufficient to ensure that large puddles, which may cause discomfort to pedestrians and may momentarily blind a driver if he suddenly strikes one, are not left. This, however, can well be done by the use of a flat cross-section tilted towards the side on which the drains are to be laid. It is so done on the *autobahnen*, and the result is that it is equally pleasant to drive on any part of the carriageway, and there is no tendency to hug the centre of the road and so to endanger other traffic. Even in a single-carriageway road the small ridge which would be formed in the centre if the water were drained to each side would be of use, because drivers would tend to avoid it, and so to keep to their own side.

(7) *Surfacing.*

This problem is as yet far from being satisfactorily solved. The essentials of a surfacing are that it should naturally have a high coefficient of friction which remains as little altered as possible under the influence of wet weather; that it should be light coloured, or at least "light reflecting"; that it should be resistant to wear; that it should not glisten seriously at night when wet; and that it should give a sweet running surface with undulations of not more than 0·2 inch.[1] There is, so far as is known, no surface which has all these characteristics, though much progress has been made of recent years, especially with concrete used both as a surfacing and as a foundation.

(8) *Curbs.*

It appears generally to be agreed that, now that carriageways are provided with good foundations, raised curbs are no longer necessary to act as abutments in the case of new roads. The roads in rural areas are better without them. In urban areas they are useful, and in fact they are the only possible means of determining the edge of the carriageway and forming a water table. In the country, however, a grass edge will serve all necessary purposes,

[1] Report of Road Research Board for year ending March 31st, 1937, p. 103. H.M. Stationery Office.

and if this is virtually flush with the road surface with no open drain nearer than 6 feet from the edge of the latter, the verge so formed provides a useful safety margin if a vehicle should run off the road.

Some means of marking the edge of the carriageway is, however, necessary, and a strip of concrete about 3 feet wide outside the carriageway proper and made of a dark-coloured concrete has much to recommend it. This is the German method, and with these strips there is no doubt as to where the edge of the carriageway is. Where a curb is used, a coloured curb would be helpful. Where curbs are used in the country they should have splayed front faces, but in towns the same considerations do not apply.

(9) *Footpaths.*

Footpaths should always be set as far from the carriageway as circumstances will allow. In the country, if it is possible to place a hedge between the path and the vehicles, a great advantage is secured, and the arrangement adds both to the appearance of the road and to the comfort of its users, for the pedestrian does not get splashed, and he cannot readily step carelessly off into the path of traffic.

(10) *Paths for Cyclists.*

The Minister of Transport recommends that these should normally be 6 feet wide, and should be increased by units of 3 feet as the traffic warrants it. It is perhaps early to offer any opinion upon the adequacy or otherwise of a width of 6 feet for a cyclists' path, but it is pertinent to point out that if a 10-foot track were provided, even on one side alone of each arterial road, cyclists would have little excuse for refusing to use it, and could with justice be compelled to do so. On the Continent where a cyclists' path exists, cyclists must use it, and they do so to the great contentment of all parties.

(11) *Road Junctions and Their Control.*

The roundabout is both economical and effective. It seems to be questionable, however, whether it is necessary in the interests of safety to make the ordinary roundabout as restrictive as is suggested in the "Memorandum on the Lay-out and Construction of Roads"[1] (Diagrams 3–8). The dimensions proposed bring a heavy strain on to the steering of the vehicle, and this stress might perhaps be somewhat reduced without serious detriment to the safety of traffic.[2]

[1] Footnote (1), p. 504.

[2] It would be of assistance to heavy loads if a straight-through opening were left in each roundabout. This opening would normally be closed and would be used only for the occasional special load which has great difficulty nowadays in negotiating the ordinary roundabout.

Flying junctions have not yet appeared in this country. They will shortly do so, and will, it is certain, be found to be of immense benefit, for it is only necessary to have had some actual experience of them to realize their value.

(12) *Surface Dressing.*

It is to be hoped that the day is not far distant when the need for surface dressings will disappear and the carriageway material will itself be such that it can provide all the qualities which at present can be provided only (except to some extent by concrete) by a process of "gumming" a layer of small stone on to the carriageway.

Tests appear to show that the best size of stone chippings is from $\frac{5}{8}$ to $\frac{1}{4}$ inch,[1] and they also indicate that, with suitable stone and mixtures, and adhesion, very high coefficients of friction can be obtained and well maintained.[2] Another most desirable characteristic for any dressing is that of being able to reflect light.

(13) *Distractions.*

What is needed is for the Government, as a beginning in the control of its trunk roads, to make a clean sweep of all signs which do not tend to road safety and to reduce its own signposting to a minimum number of signs of a standard type and thus to set an example; as far as possible these signs should not bear lettering. A one-way street sign, for example, should be of the diagrammatic Continental type, so that the driver need not read any remarks.

(14) *Refuges and Pedestrian Crossings.*

It is becoming the practice to illuminate refuges,[3] but it is doubtful whether this is of any assistance to drivers of vehicles. It obviously is not necessary to illuminate the refuges from the point of view of pedestrians, and the present lights tend to add to the confusing multiplicity of lights and signs. The type which is fitted with reflectors, and not too great a number of them, has much to recommend it.

The pattern of refuge which has now been evolved and which allows the pedestrian to cross it "on the flat" is not capable of material

[1] F. C. Cook, "Road Design and Road Safety," Journal Inst. C.E., vol. 4 (1936–37), p. 181. (December 1936.)

[2] Report by Messrs. C. E. Boast, H. F. Gilbe, W. H. Glanville, and B. C. Hammond to the Eighth International Road Congress, 1938.

[3] Ministry of Transport Memorandum on the Lay-out and Construction of Roads. No. 483 (Roads), p. 16. H.M. Stationery Office, 1937.

improvement, but it is a matter of some interest to consider whether the guard posts on refuges could not be made of an elastic material.

As for pedestrian crossings, enough experience has been gained to suggest that the "Belisha beacons" have little value and do not improve the appearance of the streets. From the motorist's point of view a crossing is most readily seen if the carriageway is coloured yellow, and the best arrangement will probably be to have the crossings built in coloured concrete blocks, as these have a high coefficient of friction.

Until the Government is prepared to insist that pedestrians shall use the crossings provided for them it will be necessary to do everything to tempt them on to the crossings. Pedestrians will never keep to a crossing when there is a large volume of pedestrian traffic and little room for it to accumulate on the pavements to wait for the traffic to stop of its own accord. In such cases guard rails are of limited use, and the only satisfactory alternative would seem to be pedestrian-operated lights.

The Author is doubtful whether pedestrians will ever to any great extent use subways to which access is gained by steps, but ramped approaches might be more successful.

(15) *Aids to Driving in Bad Weather.*

Weather in which design can aid safety is chiefly that in which visibility is bad. Snow and ice conditions are not likely to be assisted by design, except that, in areas where heavy snowfalls are common, hedges increase the trouble due to drifts, and open roads are better.

The driver who is suffering under the handicap of bad visibility is greatly assisted by white or yellow lines along the axis of the road. There is probably much to be said for the development of the principle (which can at present be used only in concrete roads) of making each traffic lane in a distinctive colour, always provided that the difference in colour is sharp. At roundabouts and junctions a good deal of help can be given by painted lines to guide drivers into and out of side roads.

Other Problems of Design Peculiar to Cities.

The streets of a great city are an intractable problem. They exist, their wholesale alteration would be expensive to a degree. The chief need of most cities is the provision of by-passes or circular roads around them, and some easy means of ingress and

TABLE I.—STATISTICS AS TO THE DEVELOPMENT OF THE MAIN R

(1) Annual receipts from taxation of vehicles (including some small miscellan sums).
(2) Sums abstracted from vehicle taxation receipts by the Treasury.
(3) Real income of the Road Fund.
(4) Annual expenditure of Local Authorities upon highways, including, gener loan charges, but excluding loan expenditure.

Year.	Receipts. (1)	Sums abstracted by the Treasury. (2)	Real income of Road Fund = (1)—(2). (3)	Expenditure upon highways. (4)	Grants and loans for new roads. (5)
	£	£	£	£	£
1909–10	290,702			16,841,149	
1910–11	867,493			17,889,962	
1911–12	1,232,924			18,471,582	4,928
1912–13	1,229,003			19,072,791	60,578
1913–14	1,481,398			20,082,489	22,920
1914–15	1,620,974	Bulk of in-		20,260,684	452,082
1915–16	808,403	come		18,134,104	4,477
1916–17	152,902	abstracted		16,902,535	2,035
1917–18	173,002	for war pur-		17,318,052	895
1918–19	194,906	poses		18,424,266	102
1919	{ 129,763 8,250,000§			29,079,139*	92,072
1920	9,432,302			42,703,879	522,554
1921	10,212,458			46,161,055	2,720,817‡
1922	11,523,831			45,164,896	1,590,463‡
1923	13,313,334			46,713,831	4,184,590‡
1924	15,364,407			51,763,340	3,925,403‡
1925	17,233,238			55,156,916	669,945
1926	19,032,682	7,000,000	12,032,682	56,256,565	667,863
1927	23,456,378	12,000,000	11,456,378	58,959,425	578,787
1928	25,521,052	4,226,067	21,294,985	58,091,315	578,319
1929	26,301,965	4,920,467	21,381,498	57,762,368	557,120
1930	27,825,174	4,926,041	22,899,133	75,715,000	467,604
1931	28,134,723	4,961,000	23,173,723	77,509,000	54,172
1932	28,431,766	5,000,000	23,431,766	63,947,000	400,653
1933	29,201,133	5,200,000	24,001,133	60,352,000	1,118,276
1934	32,587,589‖	5,100,000	27,487,589	61,125,000	1,093,523
1935	30,480,176†	4,977,000	25,503,176		1,638,289
Totals	£364,483,678	£58,310,575		£1,070,858,343	£21,408,467

* Ireland omitted.
† Certain reduced levies.
‡ Chiefly Unemployment Grants, work executed by the Ministry of Transport excluding Trunk Road Programme.

exit thereto. In the case of London and the very great cities the values of land are such that it is well to consider the possibilities of overhead and underground roads. The latter method introduces very difficult problems of entrance, exit, and ventilation, but the overhead road has already been used on a large scale.

STEM OF GREAT BRITAIN SINCE THE INCEPTION OF THE ROAD BOARD.

(5) Grants specifically for new roads.
(6) Grants for "Trunk" and "5-year" schemes.
(7) Population as estimated by the Registrar-General.
(8) Numbers of mechanically propelled vehicles taxed.
(9) Mileages of classified roads.
(10) Vehicles per thousand persons.

Ref. No.	Grants made for "trunk" and "5-year" schemes. (6)	Population (in thousands). (7)	Number of vehicles (motor) taxed. (8)	Classified roads in miles. (9) Class I.	Class II.	Vehicles per thousand of population. (10)
	£					
1		44,519	113,877			2·55
2		44,916	151,577			3·37
3		45,268	192,635			4·25
4		45,436	242,062			5·33
5		45,648	306,860			6·72
6		46,048	322,221			7·00
7		44,333	345,874			7·80
8		43,710	277,022			6·34
9		43,280	188,728			4·30
10		43,116	268,518			6·20
11						
12		44,599	549,148			12·3
13		46,472		22,188	14,399	
14		47,123	873,665	22,756	14,645	18·5
15		44,325	979,000	23,229	14,739	22·0
16		44,550	1,141,400	24,047	14,638	25·6
17	494,219	44,866	1,335,600	24,328	14,930	29·8
18	2,812,134	45,014	1,547,000	24,552	15,624	34·3
19	1,545,507	45,185	1,729,000	25,111	15,682	38·3
20	723,042	45,394	1,898,500	25,139	15,686	41·8
21	818,553	45,580	2,036,000	25,528	15,747	44·5
22	2,311,738	45,685	2,172,800	25,996	15,805	47·5
23	14,077,513	45,878	2,260,500	26,417	15,924	49·4
24	5,432,127	46,082	2,196,100	26,513	16,482	47·6
25	1,453,032	46,346	2,219,220	26,585	16,644	47·9
26	200,888	46,533	2,282,014	26,663	16,774	49·0
27	277,426	46,680	2,395,317	26,779	16,837	51·3
28	265,962	46,885	2,553,975	27,015	16,855	54·6
29	£30,412,141					

§ A special Exchequer grant of £8,250,000.
|| Taxation reduced to 15*s*. per h.p.

CONCLUSION.

It is the definite opinion of the Author that a framework of motor roads adapted in their detailed characteristics for Great Britain, but primarily designed for reasonably high-speed motor traffic, is necessary and should be begun.

There are, further, two conclusions, upon policy as apart from design, with which the Author wishes to close. Firstly, the problems of the movement of highway traffic are increasingly tending to resemble those of railway traffic and to require the same species of guidance in the interests of safety; and, secondly, as a contributor to the discussion on Mr. Cook's Paper [1] observed, it is "difficult to believe that the demand for improved facilities [that is, motor roads], from all road users could be resisted much longer in Great Britain."

APPENDIX I.

Road Fund Finance.

The figures in Table I have been abstracted from the Reports of the Road Board and the Minister of Transport, and are believed to be substantially correct. It will be observed that not until after the war was a material income raised by the taxation of motor vehicles; that from 1910 to 1935 they have contributed in direct taxation some £365 millions of money; that the total expenditure upon the roads in that period has exceeded the enormous sum of £1,070 millions; that the Treasury began to acquire control of the Road Fund at about 1926; and that its abstractions amounted by 1935 to £58 millions.

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[1] *Ibid.*, p. 205.

Section III.—ROAD ILLUMINATION.

By Leonard John Davies,[1] M.A., B.Sc., and George Sail Lucas,[1] M.I.E.E.

(*Abridged.*)

Introduction.

Progress in road design continually extends the situations that call for an entirely new standard of road lighting. Methods of obtaining the desired standard are technically available and economically possible, but the application of these methods needs considerable extension in the future, if it is to keep pace with modern road works. A high mileage of road which should be lighted to modern standards has no lighting at all, or is lighted to an out-of-date standard.

The total annual cost of providing and maintaining modern lighting may be taken on the average to be from £300 to £400 per mile. This figure is small in comparison with the cost of the roads and of the vehicles using them, and it only seems logical that the main arterial roads, at least, should be equipped with efficient road-lighting installations.

All who were interested in progressive motor travel conditions noted with satisfaction the action taken by the Minister of Transport in June, 1934, in setting up a Departmental Committee on Street Lighting, under the chairmanship of Major F. C. Cook, C.B., D.S.O., M.C. The Committee, in its final Report issued in August, 1937, considered the function of street lighting as covered by four headings, namely :

(*a*) The convenience and safety of road users.
(*b*) Police purposes.
(*c*) The convenience of residents.
(*d*) Special purposes in shopping areas and important urban centres,

and classified the roads into two groups :

Group A—Traffic Routes :

Roads on which the standard of lighting should provide an ample margin of safety for all road users without the use of headlights on motor vehicles.

Group B—Other Roads :

All other roads which the responsible authority considers should be lighted.

[1] Of the Research Laboratory, British Thomson-Houston Co., Ltd., Rugby.

The Authors propose to confine their attention to the lighting of traffic routes under Group A.

Fundamental Principles of Road Lighting.

The principles underlying the successful lighting of a road are very different from those governing other branches of illumination except, perhaps, certain inspection problems. For normal indoor lighting, for example, intensities of 10 to 20 foot-candles are used, the objects are close to the observer, and are distinguished by colour and brightness contrast. On a roadway with an efficient economically possible lighting installation the intensity is less than 1 foot-candle. Where the light intensity is low and the viewpoint is distant from the observer, it is no longer possible to discriminate by colour differences, and only brightness contrasts are available. Fortunately, on a well-planned installation, objects at a distance are seen in silhouette against the bright background of the road, and the contrasts thus provided are at a maximum and independent of colour differences. The brightness of a background depends upon the quantity of light reflected from it into the eyes of an observer. In road lighting, therefore, the important factor is not so much the quantity of light reaching the road surface as the amount and direction of the light leaving the surface.

The Ministry of Transport's Committee, in their endeavour to make recommendations that would ensure a consistently high standard of road lighting, were faced with a very difficult problem. Whilst for most lighting problems it is sufficient to specify the light reaching the object, in road lighting, owing to the peculiar viewpoint, the high incident angles of the light at the road surface, and the rapid change in the reflectivity of the road surface at these angles, the quantity of light reaching the road is in itself no criterion of the effectiveness of the lighting.

A lighting installation of small light sources correctly placed with respect to the observer and the road can be more effective than an installation of larger units incorrectly placed.

The British Standard Specification, which at the moment is under revision, classifies roads by the minimum intensity at test points between the posts, but it has been recognized for some years that this specification does not ensure a standard of lighting consistent with present-day requirements. It has been suggested that a specification might be based on some measurement of the brightness-distribution—maximum, minimum, or average readings—of the road surface, since this is very closely related to visibility ; but there are

several reasons why a specification on these lines would be unsatisfactory.

Firstly, the brightness-distribution is dependent not only on the lighting equipment and the arrangement of this equipment, but also on the conditions of the surfaces to be lighted, which vary from time to time in a manner quite outside the control of the lighting engineer. Again, the visibility is concerned with the whole field of vision, including the lantern glare, and not only with the background surfaces of the road and surroundings. Owing to the changing field of vision as the observer moves along the road, the determination of the optimum brightness distribution is a very difficult undertaking.

Whilst the ultimate responsibility of the lighting engineer should be to give adequate lighting under all road surfaces and background conditions, with the variation of surface-reflectivity and backgrounds present in practice it would be unreasonable to impose on him the responsibility for a certain result when the factors are completely outside his control or liable to changes of a major nature. Therefore, although it becomes apparent that the brightness-distribution is the criterion of street lighting, it is at present impracticable to use brightness measurements as a basis for a specification. Faced with this problem, the Ministry of Transport Committee, in its recommendations, has fallen back on the time-honoured method of recommending certain arrangements of the lighting units and the quantities of light from the sources which have given satisfactory results under all reasonable conditions found in practice.

At the same time, the Committee has left as much freedom of action as possible to the lighting engineer to permit scope within the terms of the recommendations for further improvements in street lighting, and the importance and value of the trained lighting engineer in the planning of the lighting scheme is stressed.

The Production of Bright Backgrounds on the Road Surface: The Lighting Engineer's Task.

The lighting engineer's task, namely, to illuminate roads for safe and convenient night travel, is briefly to provide the motorist with a "bright" background stretching into the distance. This background must include any surface against which it is necessary to see an obstacle (or an object that might reasonably become an obstacle) and must indicate the course and contour of the road. As in all lighting methods, the surface observed should be, if possible, the brightest, but not outstandingly the brightest, object in the field of vision.

It is clear that the lighting engineer can be helped or hindered by the design and surface conditions of the road, by the nature of the vertical backgrounds, and by the economic conditions governing his layout.

The fundamental surface that has to be rendered bright is that of the road itself. Vertical backgrounds at the sides of the roads—fences, houses, walls, hoardings, etc.—present little difficulty as long as they are not too dark in colour, but the peculiar viewpoint of the road surface to an observer standing on the surface, and the high angles of incidence of the light from the lanterns, present an unusual problem in lighting as far as the horizontal background is concerned.

An observer on the road surface views the surface from 100 to 200 yards ahead at very acute angles, and the light from the lantern produces bright areas very different from those seen at most normal angles.

The lighting engineer's job is to "patch the road surface" with the "T" areas of light, so that for every normal position of an observer the road appears covered with light patches which coalesce, but do not overlap unnecessarily.

The correct positioning of the lantern is quite important. If, for example, the rows of lanterns are placed too far apart, dark areas appear down the centre of the road, or if the rows are too near (excessive overhang) the road has a very bright centre with dark curbs.

Adequate lighting of such danger points as bends, cross-roads, T-junctions, etc., depends almost entirely on the correct placing of lanterns at these points, and they should be so placed that the bright areas fall, for an observer in the normal traffic lane, directly across the danger point. On the bend the lanterns are placed on the outer radius so that the light patches fall on the road surface and not on the verge, as would be the case if the lanterns were placed on the inner radius. At cross-roads or junctions the lanterns are placed overhanging the traffic lane beyond the road junction, so that the patch of light falls across the road entry.

Since the bright areas start at a point under the lantern, the light-distribution of the lantern must be such that light is thrown at least to the base of the next lantern in the same traffic lane, and so the light-distribution is closely related to the lantern-spacing. With a staggered arrangement of lanterns, and a spacing of 150 feet, the lantern-distribution must be such that light is thrown at least 300 feet along the road in either direction, that is, the light is radiated up to angles of about 86 degrees. Some lanterns (which may be called "cut-off" fittings) limit the distribution to 70 degrees, and

these should be spaced at distances not exceeding 90 to 100 feet, if dark areas are not to appear between the light sources.

The angles of distribution of light between 70 degrees and 90 degrees are thus very important to a lighting engineer. By increasing the angle 16 degrees from 70 degrees to 86 degrees, the length of the bright area on the road is increased approximately four times, but this advantage is offset to some extent by the increased light reaching the observer direct from the light source and causing glare. Criticism was sometimes levelled against widely-spaced units of early design on this account. In more recent lanterns, however, great care is taken with the light distribution at the critical angles to ensure the optimum conditions of visibility and the minimum glare.

It would at first sight appear that "cut-off" fittings closely spaced would eliminate glare from all lanterns ahead of the observer, and thus result in optimum visibility. It is true that for certain positions of a stationary observer direct glare is absent, but as the observer moves along the road, each successive lantern produces glare at some angle, which results in what may be termed "repetitive" glare. There are sound arguments for supposing that a certain amount of direct light from widely spaced units is less distracting than repetitive glare.

Opinion is still divided as to which of the two methods results in the better lighting, but having due regard for the economic side of the question, it is the Authors' view that the use of efficient light sources in directional fittings at wide spacings results in a better standard of lighting than an installation of the same cost comprised of closely-spaced "cut-off" fittings.

Recommendations of M.O.T. Committee.

These have been dealt with in the Report under the following headings :—

(1) Mounting height.
(2) Spacing.
(3) Overhang.
(4) Siting.
(5) Power of lanterns.
(6) Distribution.
(7) Glare.
(8) Effect of the nature and condition of the road surface.
(9) Dual carriageways.

Mounting Height.

To produce a continuously bright traffic lane at a lantern-spacing

of 150 feet, the lantern cut-off must not occur before 86 degrees. If, then, the mounting height is halved, the angles would be increased to 80 degrees. Although this angular change is small, the effect on glare is very marked, and low mounting heights demand lantern-distributions which, if they are to avoid dark areas, produce excessive glare.

Similarly for cut-off fittings with a cut-off at 70 degrees and a mounting height of 25 feet, the spacing in each traffic lane should not exceed 90 to 100 feet. If the mounting height is halved the spacing should be halved, and the arrangement is uneconomical.

At the same time it has been necessary to fix a height at which the lantern can be serviced from a tower wagon, and 25 feet has been chosen as the best compromise.

Spacing.

The Report recommends a maximum average spacing of 150 feet but permits any single span to be 180 feet. It is hoped that, where cost permits, the average spacing will not exceed 120 feet.

Overhang.

The Report states that if the spacings between the rows of lanterns exceed 30 feet, dark areas appear down the centre of the road, and if the lanterns overhang the curbs for more than 6 feet, similar dark areas appear down the curbs. Varying amounts of overhang are recommended for different widths of road. While agreeing in principle with the recommendations of overhang, the Authors feel that the choice of overhang is a matter that should be left to the discretion of the lighting engineer.

Siting.

It is pointed out that central suspension is not advisable except in special cases where the road is narrow and fronted by light-coloured buildings which reflect the light back on to the road, or where trees on either side of the road make normal side mounting difficult. Side mounting is therefore recommended, and several arrangements are discussed.

Power of Lanterns.

The power of the lanterns has been expressed as the light output of the combined lamp and lantern per 100-foot run of road. A value of between 3,000 and 8,000 lumens per 100 feet is stated to be satisfactory for roads up to 40 feet in width, whilst for wider roads the requirement of the additional central lantern every three spaces provides the necessary extra light.

Distribution.

Beyond recognizing two distinct forms of distribution in the cut-off and non-cut-off type fittings, and mentioning that the distribution of light from the lantern should be designed to produce a high and uniform background-brightness, the Report makes very little reference to the lantern-distribution.

There is no doubt that, next to correct siting of the lanterns, the light distribution is the most important factor in good street-lighting. The correct sharing of light between the road surface and other backgrounds, the careful control of the light over the critical angles that affect glare, and the general shape of the distribution-curve to produce an even road-brightness are all factors of exceptional importance.

In avoiding a decision on this question, it appears that the Committee have been influenced by two factors :—

(1) Without good siting not even the best lantern will give a uniform background-brightness, and in covering siting the Committee have dealt with the most expensive part of any new installation.

(2) It would be difficult at this stage, with the varying conditions of road surface and background, to reach agreement as to what constitutes a satisfactory distribution.

Glare.

The Report does, however, attempt to control the distribution in one respect at least—in connexion with glare. The accurate control of the light radiated near the horizontal is very important, and the Report attempts to control this by stating a maximum ratio between the light intensity at its peak (occurring between the angles 70 and 90 degrees, except for cut-off fittings) and the light intensity between 30 and 45 degrees to the vertical.

Most modern lanterns meet this requirement with little difficulty, and it appears that the recommendation was framed to prevent the excessive glare from lanterns at lower mounting heights and widely spaced, which attempt to throw light to the base of the next standard.

Effect of the Nature and Conditions of Road Surface.

It is clear that the optical properties of the road surface are of paramount importance in road lighting. The Report recognizes that for some time to come the choice of road materials will be governed by non-skid, economic and such factors.

It is to be hoped, however, that there will be more and more co-operation between the research workers and the road lighting

engineers. The Authors do not think that developments are sufficiently advanced for the lighting engineer to specify dogmatically his requirements as regards road surface, but some ideal requirements as regards the optical qualities are that the surface :—

(1) Should be semi-matt and light in colour.
(2) Should be resistant to the polishing action of traffic.
(3) Should be generally similar throughout the country, and at least without sudden changes.
(4) Should not be markedly changed by re-surfacing.
(5) Should have the minimum difference between dry and wet conditions.

Dual Carriageways.

It has been suggested that lantern-distribution should be unidirectional, but whilst under ideal layout conditions this may be advantageous, difficulties arise at road junctions, crossings and bends. Some reduction in light in one direction may be possible, and the Report recommends that this should be investigated. Otherwise the lighting recommended follows the general principles for normal roads, each carriageway being treated separately.

Paper No. 5083.

"The Reconstruction of the Mocoretá and Timboy Bridges, Argentine North-Eastern Railway."

By Geoffrey Charles Blofield, B.Sc. (Eng.), Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

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Introduction.

The Argentine Eastern Railway between Concordia and Monte Caseros, which was built in about 1870, and was one of the earliest lines constructed in Argentina, is now incorporated in the Argentine North-Eastern Railway. The principal bridges were made of wrought iron, and, although built for light rolling-stock, most of them are still in service, carrying 100-ton locomotives. For many years, however, their condition has not been satisfactory, and slow-speed orders are in force over most of them.

The bridge over the river Mocoretá consisted of two 30-metre bowstring girder spans on masonry abutments and a central pier of cast-iron cylinders, the depth of the latter being unknown. The calculated strength of the main girders and deck system did not provide a satisfactory factor of safety with present-day loads, and

[1] Correspondence on this Paper can be accepted until the 15th July, 1938, and will be published in the Institution Journal for October, 1928.—Sec. Inst. C.E.

rivets were constantly working loose. The abutment-foundations were very shallow, and in 1928 the north abutment began to incline forward. In spite of attempts to anchor it back, the movement continued, and finally it was necessary to demolish it and substitute a temporary piled support and a short leading-on span.

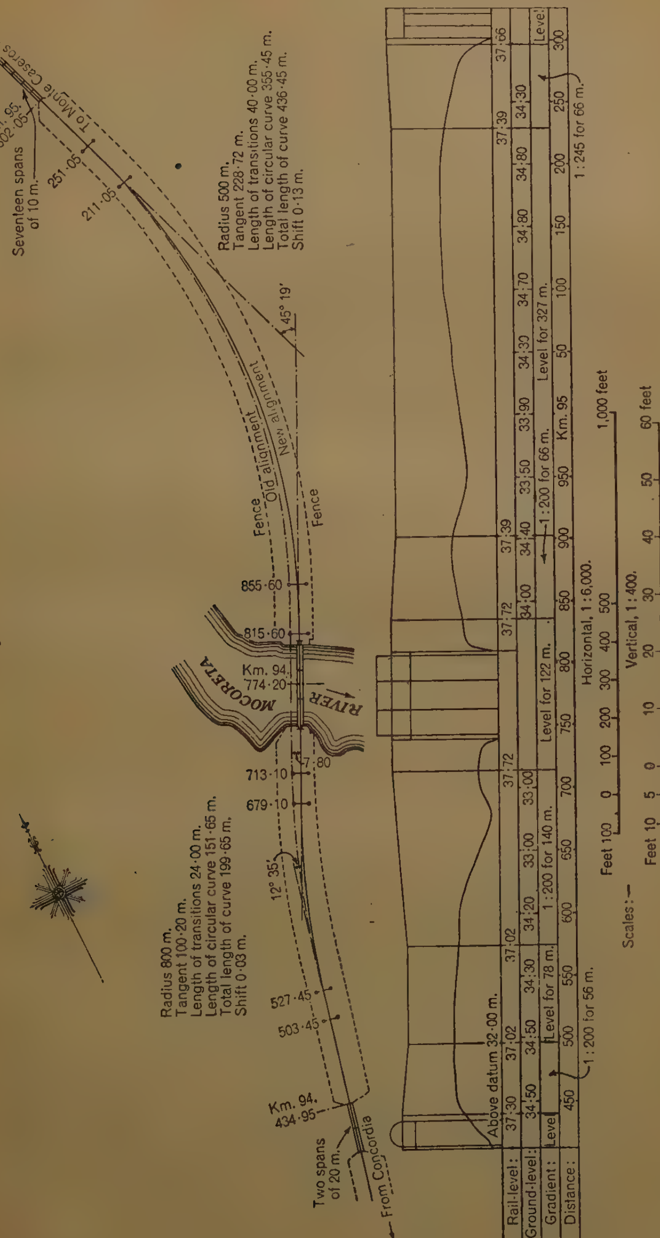
The Timboy bridge consisted of three wrought-iron bowstring girder spans each with an overall length of 20 metres, of very light construction, and its factor of safety was unsatisfactory. Rivets were constantly working loose, most of the cast-iron bearing plates were broken, and a few cracks were observed in the iron near rivet-holes. The spans were carried on masonry abutments and piers, built in coursed ashlar in cement mortar, which were in excellent condition.

The Mocoretá Bridge Scheme.

In 1934 the condition of the Mocoretá bridge called for immediate attention, and in spite of the very poor financial condition of the Company, the construction of a completely new bridge and approaches was decided upon (*Figs. 1*). Borings made on both banks and in the middle of the river indicated a stratum of hard blue clay at level 16·95 metres, or 13·30 metres below the lowest water-level and 20·77 metres below rail-level. This was overlain by sand and occasional gravel up to level 27·00 metres, with hard earth to the river-bed at about 27·90 metres. The river-banks were of clay overlain with sandy earth.

In order to minimize the cost of the new bridge, it was decided to utilize three available 20-metre plate-girder through spans, originally intended for a new extension but never utilized. Piers were to be constructed in the river, consisting of four cast-iron screw cylinders, 2 feet 3 inches external diameter, which were also available. Instead of normal abutments, reinforced-concrete pillars on a concrete slab, founded on ten reinforced-concrete piles, were to support the ends of the main spans, whilst the end slope of the embankment was to be faced with dry-stone pitching, and a short approach-span was to carry the track from the end of the bank to the main spans (Figs. 2, Plate 1). Observation of the flow through the old bridge during floods indicated little current at either abutment, although a maximum flow of about 6 miles per hour occurred in the centre of the bridge; therefore no trouble was anticipated with this type of construction. The new bridge was to be constructed alongside the old bridge, as near as possible on the downstream side, and new embankments were to be built in order to modify the curves on either side without introducing irregular curvature. It was decided to incorporate transitions in the new curves (*Figs. 1*).

Figs. 1.



MOCORETÁ BRIDGE: SITE PLAN AND PROFILE.

During periods of drought, which are not infrequent in the locality, the depth of water at the bridge-site is about 8 feet; but after heavy rainfalls, which may occur at any time of the year, the river rises rapidly to within 5 feet of rail-level. The floods usually subside slowly, and the water may remain at a comparatively high level for weeks or months. Fortunately the construction of the new bridge coincided with an 8-month drought; but the possibility of a sudden flood had to be borne in mind throughout.

The ample depth of water on the site of both piers favoured the use of a floating stage rather than of temporary piled structures for driving the cylinders, and the possibility of prolonged high water rendered this all the more desirable as work could then be continued at almost any state of the river. Large pontoons were not available, and in any case could not have been brought to the site easily; but the Company possessed two iron pontoons, 30 feet long and 9 feet wide; these were brought up by rail, and proved adequate.

A construction-camp was formed at Mocoretá station, 4 kilometres north of the bridge—the most suitable site—and a temporary siding was constructed to accommodate the ballast-train. Work on the embankments was commenced on the 1st March, 1935, and as soon as possible a siding for unloading materials was constructed on the north side of the bridge. A 5-ton locomotive crane was stationed in this siding throughout the work, for handling materials and erecting the steelwork.

Screw-Cylinder Piers.

The equipment for screwing the cylinders and driving the concrete piles included two vertical boilers, installed on the south bank of the river, which supplied steam for winches, pumps, and the piling-hammer. A Widnes steam-winch, similar to a ship's winch, with two cylinders of 8 inches diameter and 12 inches stroke, having a central drum and two capstan heads, was used for setting and screwing the cylinders. One Worthington and two Hayward-Tyler double-acting two-cylinder pumps installed on the old bridge were used for jetting. A 35-foot steel pile-frame with a 1-ton single-acting steam-hammer and a small steam-winch was used for piling. A three-cylinder petrol-driven Merryweather pump was used to supplement the steam-pumps when necessary, whilst a small petrol-driven diaphragm suction-pump was also available. A 240-cubic-foot Holman petrol-driven air-compressor was used for riveting the steelwork.

The cylinders were 2 feet 3 inches in outside diameter, in 12-foot lengths, with walls about $1\frac{1}{8}$ inch thick, and had inside flanged joints formed with twelve $1\frac{1}{4}$-inch turned bolts. The screw-blade,

5 feet in diameter, was carried on a 2-foot length of cylinder with a cutting-edge at the bottom. On completion of driving, the uppermost length was cut to the required dimensions and a cast-iron cap with a sleeve 9 inches long fitting over the cylinder was placed over the top.

Screwing was effected by means of a steel capstan-wheel, 20 feet in diameter, working on a hexagonal column of cast iron which was bolted by means of an intermediate piece to the top of the cylinder. The wheel was turned by means of two steel cables $\frac{7}{8}$-inch diameter, which were first wound on the capstan-wheel and were then pulled by means of the two capstan-heads on the Widnes winch on the south bank of the river and securely anchored back. One of the cables passed around a snatch-block secured from the north bank, so that two tangential pulls were exerted, in opposite directions, producing a turning effort with little or no bending stress. The men working on the cables at the winch soon realized that approximately equal pulls must be maintained on each cable, and they were informed from the stage when any lateral movement was noticed at the head of the cylinder.

The wheel was mounted on a steel plate secured to the cross-members of the floating stage (Figs. 3, Plate 1), which consisted of two A-frames, each mounted on one of the pontoons, carrying two transverse rolled steel joists at their apex, and the two cross-timbers mentioned above, about half-way up. Detachable transverse timbers at water-level supported the screw sections during the first stage of erection, and formed supports for a working platform, besides bracing the two pontoons. The wheel, the hexagonal column, and the cylinder-lengths were raised and moved by means of lifting tackle supported from a trolley running along the rolled steel joists. The tackle was actuated from the steam-winch and the trolley was traversed from side to side by means of a hand-winch and an endless cable.

For securing the stage in position, six mooring-cables, passing through blocks at the level of the wheel, and secured to hand-winches on the pontoons, were attached to "dead-men" set in the river-banks. Owing to a bend in the river, it was possible to secure two cables at right angles to the centre-line of the bridge, and four parallel to the bridge.

Light booms were added above the steel joists for manipulating jet-pipes and other plant. A light gangway was constructed below the joists, and ladders up to the A-frames, for the convenience of the workmen.

The position of each cylinder was located by means of a number of plumb-lines suspended from the old bridge, and little difficulty

was experienced in determining the exact position at any time. During screwing operations it was necessary to maintain a continuous watch in order to check any tendency of the cylinders to bending. The cylinders were maintained vertical by moving the stage slightly upon any deviation of the point from the correct position.

The cylinder-lengths were floated out from the south bank on a small pontoon, and were then lifted into position by the tackle on the stage. A screw point was placed on the lower transverse timbers, and a cylinder-length was bolted on. These were then lowered to the river-bed in the correct position, and a loose collar, secured to the river-banks by four cables, was dropped over the cylinder down to the screw-blade. During the early stages of screwing it was found quite easy to control the point of the cylinder in any direction by increasing the pull on one or more of these cables (Figs. 3, Plate 1). The intermediate piece, of cast iron, 1 foot in length, was bolted to the inside flange at the top of the cylinder, and had an external top flange by means of which the hexagonal screwing-column was bolted on. When the wheel was on the column screwing was commenced, the position being checked at each half-turn and corrected as necessary, until the bottom of the cylinder was well embedded in the ground.

During screwing operations the capstan wheel remained at a constant level, the hexagonal column slipping through it as the cylinder was lowered. When no more cable remained on the wheel, a small locking-piece between the wheel and the hexagonal column was withdrawn, permitting the wheel to be turned freely in the opposite direction by means of a third cable, thus rewinding the main cables.

The stage had been made sufficiently wide to permit two cylinders to be screwed without moving it; and as soon as the top of the first cylinder reached water-level the screw-point and the first length of the companion cylinder were erected and driven: meanwhile a second length was added to the first cylinder. At first the penetration was small, and several turns were necessary to lower the cylinder 1 foot; but as soon as the sand was reached the penetration increased to between 6 inches and 9 inches per turn. After the cylinder had been lowered 3 feet it was necessary to loosen and partly remove the core by means of a water-jet consisting of a 1½-inch pipe lowered from the top through the hexagonal column to within 8 inches of the cutting-edge. Water was forced through by means of one of the steam-pumps, and escaped through holes in the intermediate piece, carrying considerable quantities of the finer material. After two lengths had been driven the sand core was cleaned out by means of a diaphragm suction-pump. The 2½-inch suction-hose was

attached alongside a jet-pipe, and in this manner up to 30 per cent. of sand could be pumped up with the water, including stones 1 inch in diameter. Screwing was then resumed, and was continued until the resistance became excessive, when the cleaning out had to be repeated.

Deviation from the correct position after penetration had attained about 6 feet was corrected by means of a second jet, lowered outside the cylinder, which loosened the sand on that side, the point usually moving towards the jet. Outside jetting was also tried, with the object of reducing skin-friction during the later stages of driving; but this was abandoned, as it caused the cylinders to lose alignment.

The sand encountered was water-bearing, water rising within the cylinders about 4 feet above low-water-level. This necessitated constant pumping while each new cylinder-length was being bolted on. Below level 22 metres, large stones which had not been revealed by the trial borings were encountered in the sand, and became wedged in the bottom of the cylinders, impeding progress. Such stones were largely the cause of breakage of three of the cylinders during the screwing operation, whilst when struck by the screw-blade they also deflected the cylinders and impeded the restoration into line.

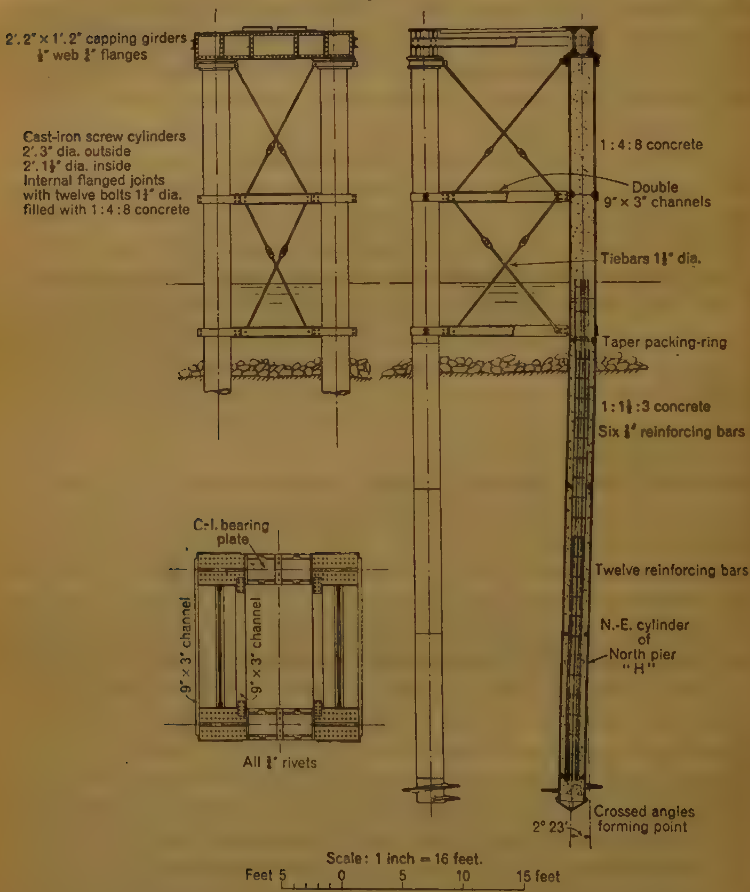
In order to prevent stones entering the screw end, this was divided into four segments by means of two crossed angles bolted in the form of a cross across the end (*Figs. 4*, p. 534, cylinder "H"). These angles were bent outwards to form a point 8 inches high, and the device proved successful in deflecting large stones from the path of the cylinder.

After various attempts, the idea of bedding the cylinders on the clay was abandoned, and they were left when further screwing involved risk of breakage. Thus, cylinder "H" (Figs. 2, Plate 1) was left at level 17·22 metres, after an attempt had been made to continue screwing, using six external jets to reduce skin friction. When the tension in the cables was released, there was about ½ inch of back-spring at the circumference of the cylinder at the top. This cylinder ran 12 inches off line, and all attempts at correction were ineffectual owing, no doubt, to the pressure of boulders against one side: it was therefore pulled over out of plumb and a tapered packing-ring was inserted at the joint just below water-level, the upper portion being then vertical and in its true position. This cylinder was filled with reinforced concrete as a precautionary measure (*Figs. 4*, p. 534).

With the experience gained on the first four cylinders the other four were driven with little difficulty, in 15 working days. A stratum of hard gravel was encountered between levels 22·40 metres and 21·70 metres. This was passed by cleaning out the core below the cutting-edge, and by external jetting.

On completion of screwing, the upper sections were cut to length by hack-saws, and were bolted in position. The cylinder was cleaned to within 1 foot of the bottom and was filled with concrete mixed on the north bank and conveyed by means of a ropeway.

Figs. 4.



DETAILS OF PIERS.

Salvage of Broken Cylinders.

The cylinders were designed to be sunk in silt, and consequently were of light section for this class of work. As mentioned on p. 533, three were broken during screwing by large stones which blocked the ends or fouled the screw-blades, although a number had already been

sunk in sand and gravel, to a smaller depth, without incident. As an indication of the size of the boulders encountered, it may be mentioned that a flat stone, 2 feet long and 18 inches wide, was actually brought up on the screw blade of a broken cylinder.

Cylinder "H" was the first to break, the fracture occurring with the cutting-edge at level 22·20 metres. This cylinder was the first to attain this depth, and at first it was thought that the screw was in quicksand as, when screwing was resumed, the top of the cylinder continued to drop, although in a series of jerks. Later it was realized that this was due to breaking up of the fractured ends.

Cylinder "G" was being driven alternately with cylinder "H" and had attained level 19·75 metres, the third section being then submerged 8 feet, when it broke at river-bed level. The fracture appeared to be due to a flaw in the cast iron, and it was concluded that lateral movement of the screwing-head had assisted by inducing bending stresses. As the fracture was apparently at the lower end of the third section, a wooden tube, 4 metres long and fitting loosely over the cylinder, was constructed, and was lowered until its bottom end was over the second length. The tube had a rubber joint at the bottom designed to close on the cylinder-wall when the external pressure exceeded the internal pressure. As soon as this was in place it was pumped out, when the discovery was made that the top flange of the second cylinder-length was also broken. A second tube was then constructed and joined to the first, making a length of 8 metres. This was fitted with a perforated pipe ring fixed on the cutting-edge, through which water was pumped, and by loading the tube and assisting with an external jet it was lowered until the joint reached the first cylinder-length (Figs. 5 (*a*) and 6 (*a*), Plate 1). Attempts were then made to pump out the cylinder to level 24 metres, or 21 feet below water-level, but these proved ineffectual as the pumps became clogged, after about 18 feet had been pumped dry, by large quantities of sand drawn in with the water entering the foot of the cylinder. Attempts were made to plug the cylinder, first with a canvas bag filled with earth and then with a wooden disk, hinged in the centre to allow it to pass the internal flanges. These devices were lowered with the tube full of water after the sand had been cleaned out, but it was not found possible to place them successfully at the depth required. A simple cylinder-pump, made from a length of 6-inch pipe, was then constructed and lowered inside the cylinder; this, together with three hand diaphragm pumps, proved sufficient to lower the water momentarily to the required level. The wooden disk was then placed by hand, and it was found possible to maintain this water-level while the broken second cylinder-length was unbolted, removed, and replaced with a sound length.

A third length was then added and the wooden tube was removed, after which screwing was resumed.

In the meantime, the screwing of cylinders "E" and "F" had been started. Cylinder "F" had attained level 19·55 metres when more boulders were encountered and it was broken at the bottom end of the second length, about 21 feet below water-level and 13 feet under the sand. As difficulties had been encountered in salving cylinder "H" it was decided to work on "F" first. The wooden tube was fitted with eight curved iron prongs to guide it over the top of the broken cylinder, and was lowered by jetting and weighting; but it was found impossible to pass it over the cylinder. A conical sheet-iron tube, 3 feet long and 3 feet 4 inches in diameter at the bottom was then fixed to the bottom of the wooden tube, and this enabled it to be successfully located over the cylinder (Figs. 5 (*b*) and 6 (*a*), Plate 1). These operations involved considerable work; but by repeated inside and outside jetting in conjunction with the removal of sand from just below the bell-mouth by means of a suction-pipe and jet lowered outside the tube, the rubber joint was successfully lowered over the first cylinder-length. Further difficulty was caused by an inrush of sand from the bottom of the cylinder when pumping was commenced, and the cylinder-pump became jammed. A filter consisting of a perforated drum covered with wire gauze (Figs. 5 (*b*), Plate 1) was fitted, but this also became choked with sand, and when an attempt was made to withdraw it, became detached from the pump. A collapsible plug of canvas over a frame of wood and iron was then devised on the principle of an umbrella, but with parallel sides extending from the tips of the ribs (Figs. 5 (*c*) and 6 (*b*), Plate 1). This was opened after passing below the flanged joint of the cylinder and proved successful in stopping the inrush of sand. It was then comparatively easy to pump out the tube.

The discovery was made, however, that part of the upper flange of the first cylinder-length was also broken, only four holes remaining sound. As it was now necessary to remove the cylinder and screw entirely, steel cables were secured to the sound bolt-holes and attached to the lifting-tackle on the stage. While six water-jets, supplied from four force-pumps, were lowered simultaneously to the level of the screw-blade, the tackle was gradually strained until a pull of 10 tons was being exerted. The cylinder was lifted slowly in this manner for a distance of 9 inches, when the remaining piece of flange suddenly gave way. After removal of the broken piece, the collapsible plug, and the filter, attempts were made to effect a hold on the bottom flange of the cylinder. A grapnel, fashioned from a piece of rail with steel cables fastened at the centre, and attached to a jet-pipe for lowering, was then constructed (Figs. 6 (*c*),

Plate 1). It was found impossible to lower this through the sand in the cylinder; but eventually it was placed below the flange by attaching it loosely to the end of a 20-foot rail (Figs. 5 (*d*), Plate 1). The cables were then attached to the lifting tackle and pulled, and the cylinder was moved a few inches, when the grapnel slipped, evidently because it had not been properly engaged. Attempts to lower it again proved fruitless, owing probably to pieces of stone or iron inside the cylinder.

An arrow-headed grapnel was then devised and made from scrap iron (Figs. 6 (*d*), Plate 1). This consisted of two diamond-shaped $\frac{3}{4}$-inch plates forming the central body and fixed to the end of a rail. Between these plates two arms, also made of $\frac{3}{4}$-inch plate, were mounted on pivots, so that they could close against the rail or could open to form a cross-arm 1 foot 10 inches in length, which would engage in the flange-joint of the cylinder. The arms were opened by means of light wire ropes. This grapnel was attached to a second rail, making a total length of 35 feet, and was then lowered by jumping it and simultaneously lowering a jet alongside until it had passed the flange-joint, when the arms were opened (Figs. 5 (*e*), Plate 1). A pull of 12 tons was then exerted while eight water-jets were lowered to loosen the sand, and the cylinder was slowly withdrawn. It was on the blade of this cylinder that the large stone mentioned on p. 535 was brought up.

Attempts to lower the wooden tube over cylinder "H" had proved unsuccessful owing to the pressure of pieces of broken cylinder, which had fallen crosswise over the broken end. These were embedded in 6 feet of sand, and all attempts to move them, by grappling with hooks of various kinds, specially-made grappling-irons, jetting alongside, and even sucking sand from alongside and below, resulted in failure. It was impossible to excavate the sand, as any hole immediately became refilled. A two-jawed dredging-grab (of $\frac{1}{2}$ cubic yard capacity) was then procured and was slung from the lifting-tackle on the stage, lowered in the open position to the river-bed, and sunk gradually by jetting all round, until it rested on the pieces of iron (Figs. 5 (*f*), Plate 1): it was then closed and pulled up with the lifting tackle. After several ineffectual attempts, one large piece of iron was removed, whilst later a second piece was pulled up. The tube was then lowered over the cylinder, the arrow-headed grapnel was lowered and engaged with the bottom joint-flange, and the cylinder was pulled out in a manner similar to that adopted for cylinder "F" (Figs. 5 (*g*), Plate 1).

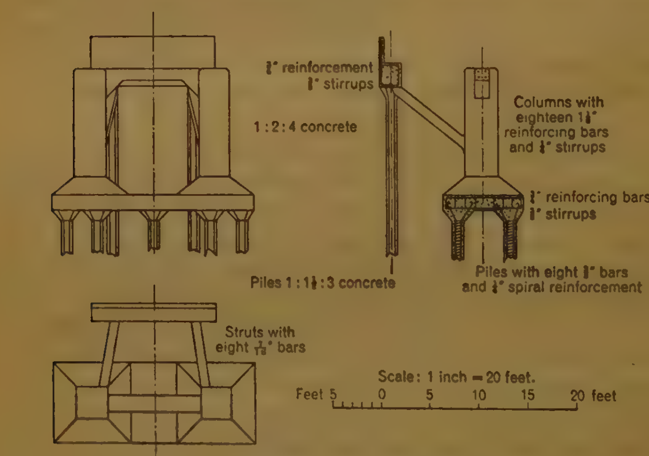
As soon as the cylinders had been driven, the horizontal bracing was cut to length and drilled. The diagonal tie-bars had been supplied with one blank end, which was now forged up and drilled.

The lower horizontals and diagonals were then fixed on the cylinders above water-level, and were painted. They were attached to the cylinders by means of collars, one flange of each collar being left with the bolts slack. When ready, the bracing was slipped down the cylinders to its correct position under water, and the loose bolts were tightened by means of a long-handled box spanner with ratchet attachment. The employment of a diver was thus rendered unnecessary. The remainder of the bracing was then placed, after which the cast-iron caps and the capping girders were erected by means of the floating stage.

Abutments.

The construction of the two abutments proceeded on the river-banks while the cylinders were being driven. As no suitable site free from flooding was available at the bridge, the twenty-four

Figs. 7.



DETAILS OF ABUTMENTS.

concrete piles were made at the station and conveyed by rail. No difficulty was experienced in driving them, except that the driving was very hard and at times jetting had to be resorted to. The reinforced-concrete abutments (*Figs. 7*) were then constructed, the reinforcement being built up beforehand and placed by crane, in order to save time and to reduce the risk of the work being flooded by a sudden rise of the river.

Plate-Girder Spans.

In order to avoid the necessity of falsework, the plate-girder spans were erected and riveted on the new embankment at the north end. A siding was constructed on the new bank, to serve for unloading the materials and later for the erection of steelwork with the 5-ton crane. On the line of each main girder a 2-foot-gauge track was laid, which provided a base for the erection work and enabled the girders to be moved forward on trolleys when completed.

Only the main girders of span No. 1 were erected and riveted. These were then moved forward to overhang the abutment, providing room for the erection of span No. 2, which was riveted up complete with its floor system. The main girders of spans Nos. 1 and 2 were then joined with temporary splice-plates and angles on the top flange and packing-plates between the ends of the bottom flanges (Figs. 9 (*c*), Plate 1). This joint was designed so that the stress in the rivets would not exceed 8 tons per square inch when the girders of span No. 1 were cantilevered out to overhang the first pier. The joints were never actually subjected to this stress. Span No. 3 was also erected and riveted complete, but was not yet joined to span No. 2. At this stage spans Nos. 2 and 3 were painted. Spans Nos. 1 and 2 together were then supported on rollers at the abutment (Figs. 9 (*c*), Plate 1), and on trolleys under span No. 2 (Figs. 9 (*b*), Plate 1), and were pulled forward by means of the steam-winch and tackle until the leading end had traversed half the distance to the north pier (Figs. 8 (*a*), Plate 1). At this stage the tackle was moved to the top of a 20-foot A-frame, mounted on the pier, which was designed to support 10 tons of the weight of the leading end, thus reducing considerably the load on the temporary joint. Care had to be taken to avoid the production of tension in the bottom flange of the joint, and the position at which the tackle could be transferred to the A-frame had therefore to be determined by calculation. As the only firm bearing at the abutment consisted of the tops of the concrete pillars, the rollers had to be maintained exactly at this point. Rails 1 metre in length were fixed on the tops of the pillars, and a track made of channels was secured to the bottom flange of the girders, the rollers working between these. The ends of the rails were cut to a slope to facilitate the entry of new rollers. As soon as a length of the channels had passed the rollers it was removed and placed behind; thus only two lengths were required for each girder.

When the leading end reached the north pier it was supported on rollers, running on rails, 4 metres long, laid along the top of the pier. Short lengths of channel ran on the rollers, and the girders were supported on these by packing. When the rollers reached the south side of the pier, the girders were jacked up and the rollers were

removed to the north end. The position of the rollers on the pier had to be taken into account when calculating the stresses at the temporary joint (Figs. 8 (*b*) and 9 (*d*), Plate 1).

As the erecting tracks had sunk, owing to the softness of the earth of the new embankment, it was necessary, as soon as the girders were moving on two rigid supports, to devise a support at the rear end which would remain at constant level. This consisted of a lever under the end of each main girder, running on a trolley (Figs. 9 (*a*), Plate 1). A leverage of 10 to 1 was obtained, using two rails for levers; and by loading these rails with sleepers an upward lift ranging up to 13 tons per girder could be obtained easily. As the trolleys passed over inequalities in the tracks the levers would rise or fall, but the supporting force would be constant. The load on the levers was reduced gradually as the girders moved forward, in order to avoid tension in the bottom flange at the temporary joint. Spans Nos. 1 and 2 were moved forward until the leading end was 5 metres from the south pier (Figs. 8 (*c*), Plate 1), when the tackle was transferred to the A-frame, now mounted on that pier. They were then pulled forward to the pier (Figs. 8 (*d*), Plate 1), where rollers were placed under the leading end. Span No. 3 was now pulled up and riveted temporarily to span No. 2, the trailing end being then supported on the levers. The process was repeated until the girders were resting over their final positions, when the temporary joints were removed and the spans were jacked down on to their bearings. The floor of the first span was then erected and riveted by means of a hand winch mounted on trolleys running along the main girders. A working platform was also suspended from these trolleys. A similar arrangement was adopted for the Timboy bridge (see p. 543). The cross girders and rail-bearers were transferred from a wagon to the hand winch by means of the crane standing on the second span.

Track.

New 75-lb. B.S. rails on hardwood sleepers were laid along the new bank, which was consolidated as far as possible by running a locomotive over it. A short length of the old main line at each end of the deviation was also renewed, and on Monday, 11 November, these were slued over and connected to the new track, bringing the deviation and the new bridge into service.

The Timboy Bridge Scheme.

Nature of the Problem.

The reconstruction of the Timboy bridge presented an entirely different problem, and is worthy of notice chiefly on account of the

unusual method adopted in carrying out the work. It was proposed to cap the existing masonry abutments and piers with reinforced concrete, on which would be carried the three new plate-girder spans, of similar type to those used at Mocoretá. These spans were available for use, but had to be shortened; this was considered preferable to the purchase of new steel. The two end spans were cut off as closely as possible to the end cross girders, but one end bay of the central span had to be completely modified. This work was done on the site prior to erection.

Like the Mocoretá river, the Timboy river is subject to sudden rises at any time of the year, but the water subsides rapidly to its normal level and the floods seldom exceed a few days in duration. The river-bottom is sandy, but is covered near the bridge-side by considerable quantities of stone rubble deposited to prevent scour. The depth of water ranges between 6 feet and 12 feet under the bridge, the river extending roughly between the centres of the end spans. The abutments are built on the river-banks, with fairly shallow foundations (Figs. 10, Plate 2).

The erection of a temporary bridge and deviation to carry traffic during the reconstruction was considered, as also was the erection of the new spans alongside on temporary staging, to be rolled into place. A maximum period of 24 hours between trains, once per week, was available, and after careful consideration it was decided that this would afford ample time in which to remove one old truss and build up a new span in its place. Thus very little temporary staging was required, whilst no piling was necessary. The scheme involved lowering the old spans between the piers, lifting in the new main girders, building up the floor system, and laying a temporary track, which would be replaced by the permanent sleepers and rails when riveting was completed. Each new span was to be completely riveted and painted before the next was renewed, so that one set of working stagings would suffice. The old trusses, with track, weighed 20 tons, and each new main girder 18 tons; two 10-ton cranes were therefore required for the operation.

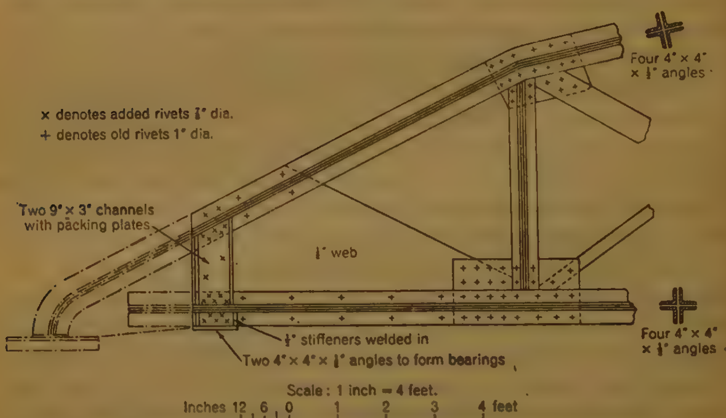
A short temporary siding was laid on the south side of the bridge, the turnout being placed close to the abutment. The ground here was high so that only a small quantity of earthwork was required, and the siding ended at ground-level. A small area of land was rented for the purpose, and was used also for the workmen's huts. This siding was necessary for unloading materials, and the level ground at the end was utilized for the erection of the main girders.

Removal of the Old Trusses.

In order to lower the bowstring girders between the abutments

and the piers, it was necessary to cut off the ends, and thereafter, until the renewal took place, the girders were supported on temporary trestles in front of the abutments and piers. Wooden supports consisting of four 10-inch by 10-inch timber posts, mounted on mud-sills and capped with a 10-inch by 10-inch rolled steel joist, were erected in front of each abutment, after the ground had been cut away sufficiently to permit the old spans to be lowered clear of the new girders. The piers, which had cut-water ends, were built on rectangular bases, the tops of which were just below water-level. These were used to support the temporary trestles, 11-inch by 11-inch rolled steel joists placed across each end of each pier supporting a plate girder, 26 inches high and 14 inches wide, which ran the length

Fig. 12.



END OF OLD MAIN GIRDER AS STRENGTHENED AND CUT.

of the piers. On this stood four 12-inch by 12-inch timber posts, capped with a 10-inch by 10-inch steel joist and secured by bolts passing through the masonry. In order to counteract any tendency for the 11-inch by 11-inch joists to tip up when one support was loaded, a third beam was fastened between the plate girders, passing through the jack-arch of the pier and wedged down from the soffit of this arch (Fig. 14, Plate 2).

The old bowstring girders were stiffened at each end by adding vertical channels to the end web-plate, forming a short end-post. A bearing was made by riveting to the bottom flange of the girder two angles back to back (*Fig. 12*). The spans were lifted slightly and blocked up on the supports, and the ends were cut off with a cutting torch. This enabled the masonry of the piers to be cut away and the reinforced-concrete slabs to be constructed; the latter were made with quick-setting cement.

Renewal of Spans.

In the meantime the main girders for the first span had been erected and riveted as far as possible, one on each side of the siding, with the aid of a 5-ton locomotive-crane. They were then supported from two bogie flat wagons by means of a 11-inch by 11-inch steel joist bolted transversely to the top flanges at each end of the girders, the joists being supported on packing placed on the centre of each wagon, so that the bottom of the girders was slightly above rail-level (Fig. 13, Plate 2). The wagons were drawn out over the bridge, the girders passing outside the old truss until they could be blocked up on the ends of the concrete caps (Fig. 14, Plate 2). The joists and wagons were then removed.

As soon as possession of the track had been obtained for the 24-hour period, two locomotive cranes of 10 tons and 15 tons capacity respectively, each with a wagon of materials attached, were placed one at each end of the span to be changed. The track was cut at each end of the span by lifting out a short length of rail, the old span was lifted slightly, and the capping-joists of the temporary supports at each end were removed. The old span was then lowered on to the transverse plate girders below, and the new main girders were lifted, one at a time, and placed on their bearings, set 1 inch wide to facilitate the placing of the cross girders. On the top of each main girder rails, running the whole length, had been secured. On these rails were placed four two-wheeled trolleys, from which angle-iron hangers were suspended on the outside of the span, and to these hangers were fixed wooden platforms spanning the width of the bridge below the girders (Fig. 15, Plate 2). On the same trolleys were placed pairs of rails transversely across the top of the span, which carried a hand-winch on each pair. Erection of the floor system was then begun, commencing with the middle cross girder. Each member was picked up in turn by one of the cranes, and passed to the corresponding hand-winch, which was run along to the correct position; the girder was then lowered into place and bolted up. As soon as the whole floor system was in place, the main girders were drawn together by tightening the bolts holding the cross girders. A temporary track, prepared in short lengths, was laid by the cranes, and the transverse rails and hand-winches were removed. The bridge was now ready to carry traffic.

Riveting was continued under traffic, with the aid of the same travelling platforms, slightly modified (Fig. 16, Plate 2). Air was supplied from the Holman compressor installed at one end of the bridge, whilst rivets were heated below the bridge on the platforms. A few rivets in the bottom flange at each end of the girders could be put in only by lifting the spans. This operation was carried out

between trains when possession could be obtained for at least 4 hours. At first jacking was tried; but it was found to be quicker to lift the spans by means of two levers, for which the 40-foot plate girders previously used for the temporary supports were employed. These worked on a fulcrum on the end cross girder of one span, and lifted the adjacent span by means of a rolled steel joist bolted across the top flanges. The plate girders, weighing 3½ tons, could be placed quickly with the 5-ton crane, and lifted the spans by their own weight.

Electric lighting was installed to provide adequate illumination for the night work. Current was supplied by a small generating-set. For the whole period during which the trusses were on the temporary supports, and until the permanent track was laid, a slow-speed order was in force over the bridge, and all trains were passed by hand signals, at a walking pace.

Each span was painted as soon as the riveting was complete, after which the travelling platforms and trolleys were removed ready for use on the next span. The permanent sleepers and rails were then laid, whilst a short length of track at each end of the bridge was also renewed and reballasted. The old trusses were suspended from the new girders as soon as the work allowed, and were then dismantled by means of an oxy-acetylene torch, the pieces being loaded by the 5-ton crane standing on the bridge.

Work was started on the temporary siding on the 10th December, 1935; the first span was changed on the 1st February, 1936, starting at 6 p.m., and the bridge was ready for traffic by 7 a.m. on the 2nd February. The second span was changed on the 21st February, between 3.30 p.m. and midnight, and the third span on the 13th March between the same hours. The work was completed on the 7th April, 1936.

COSTS.

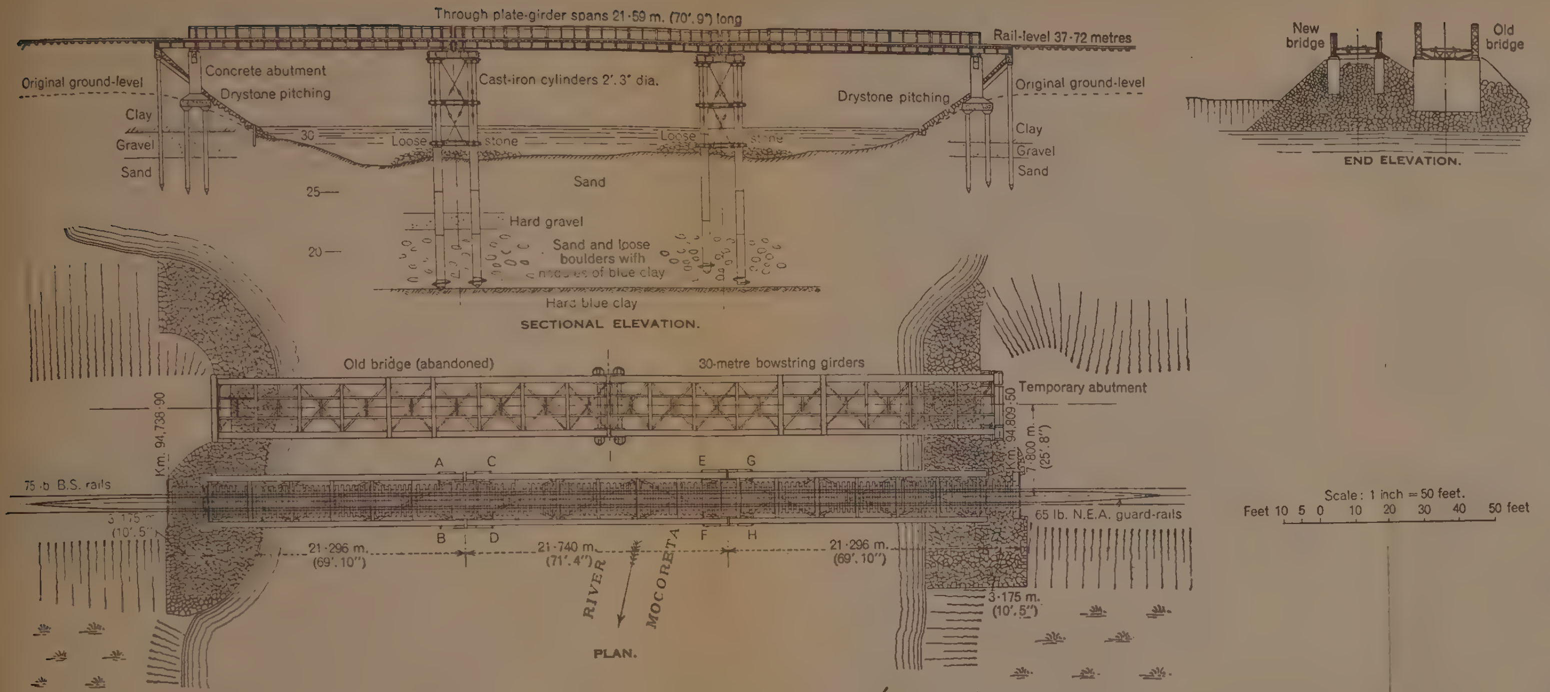
Both works were carried out by the Railway Company's own forces, supplemented by temporary labour, under the Author's directions. The cost of the Mocoretá bridge and approaches amounted to £9,300 (at current rate of exchange), including £1,250 for the screw-cylinder piers, £850 for the concrete abutments, £4,400 for the steelwork, and £2,800 for earthworks and track. The cost of the Timboy bridge was £5,050.

The Author wishes to express his thanks to Mr. R. F. Williams, O.B.E., M. Inst. C.E., Chief Engineer of the Entre Rios and Argentine North-Eastern Railways, for permission to present this Paper.

The Paper is accompanied by three sheets of diagrams, from which Plates 1 and 2 and the Figures in the text have been prepared.

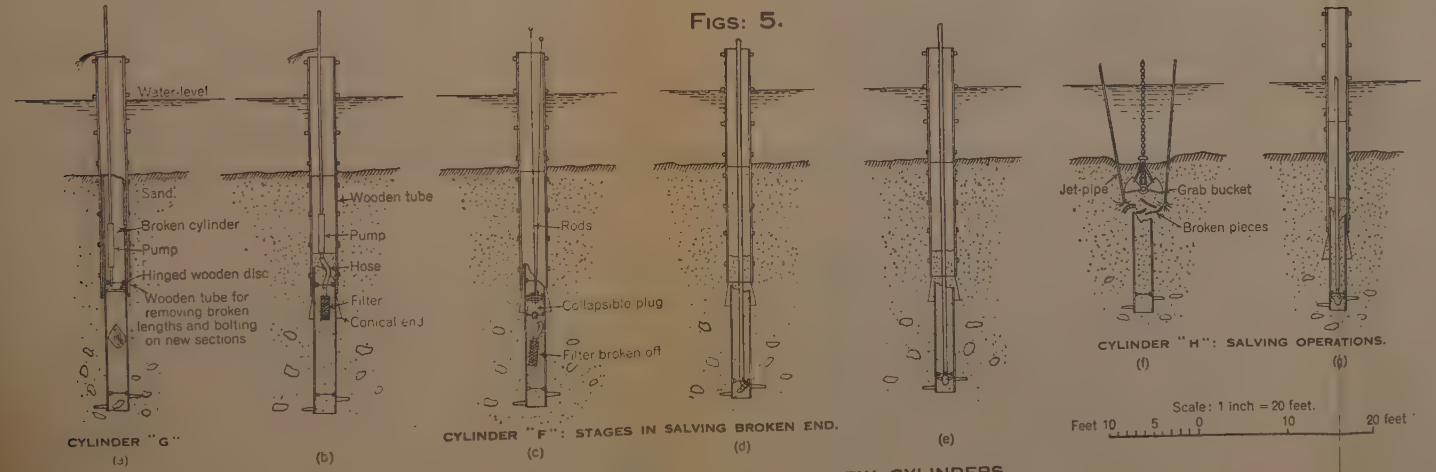
THE RECONSTRUCTION OF THE MOCORETÁ AND TIMBOY BRIDGES, ARGENTINE NORTH-EASTERN RAILWAY.

Figs: 2.



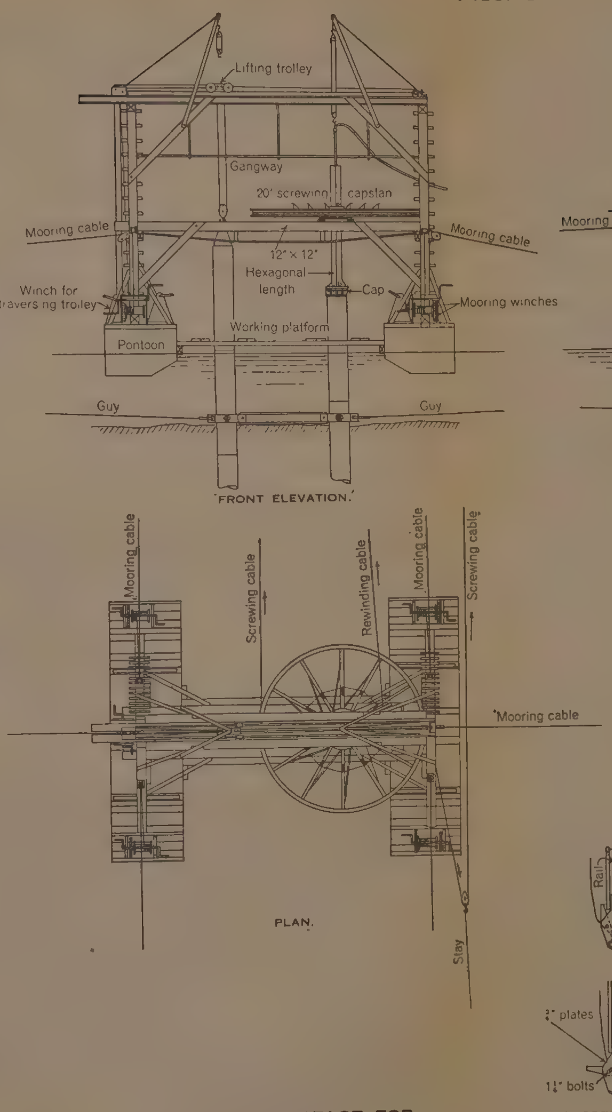
RECONSTRUCTED MOCORETÁ BRIDGE.

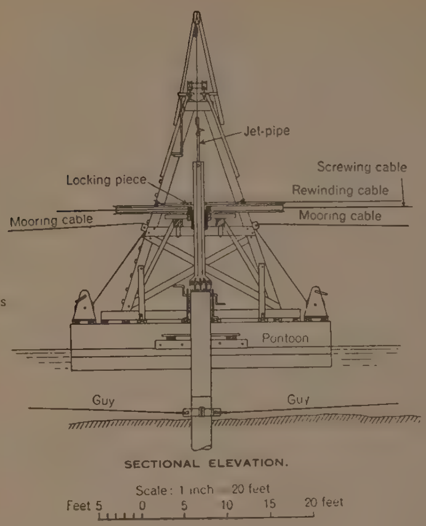
Figs: 5.



SALVING BROKEN CAST-IRON SCREW CYLINDERS.

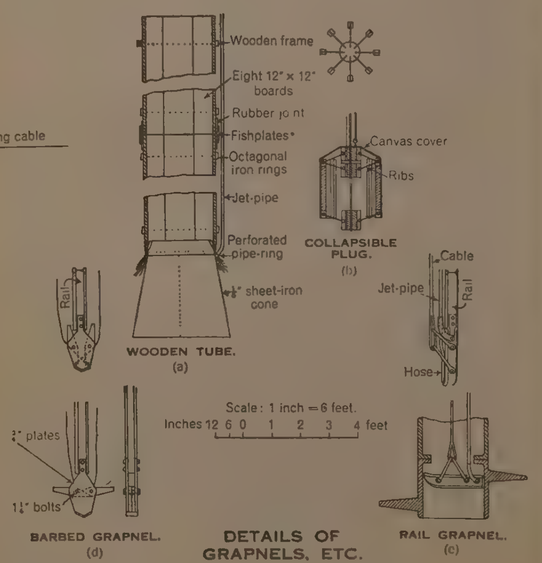
Figs: 3.





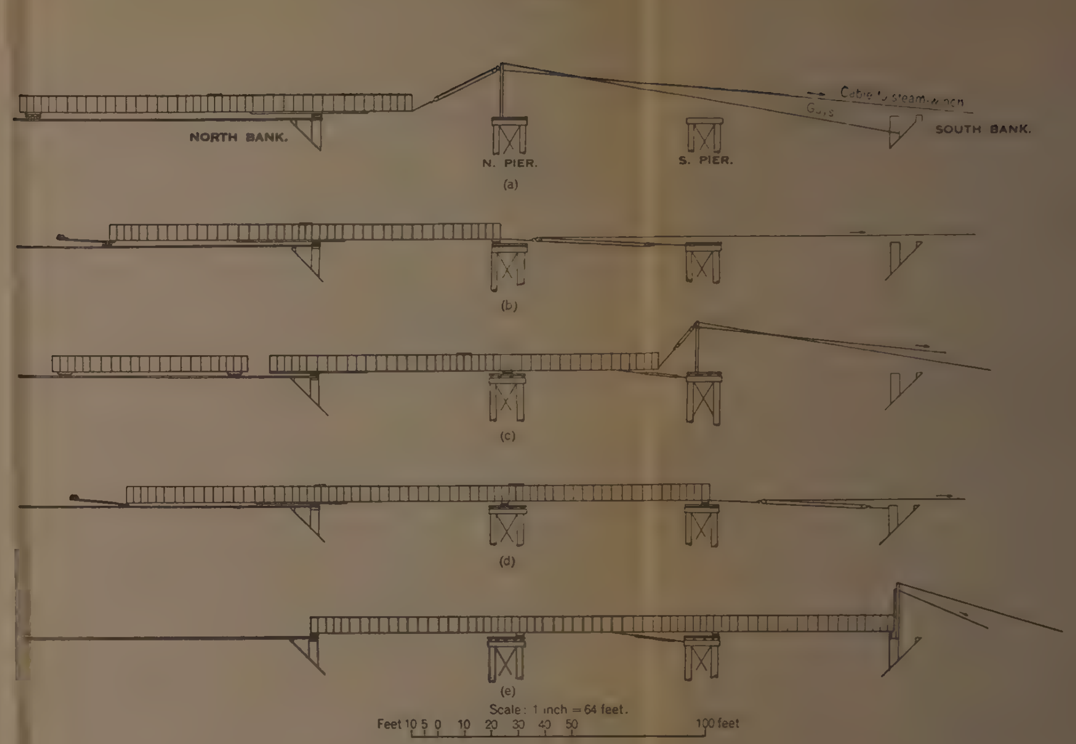
FLOATING STAGE FOR DRIVING SCREW CYLINDERS.

Figs: 6.



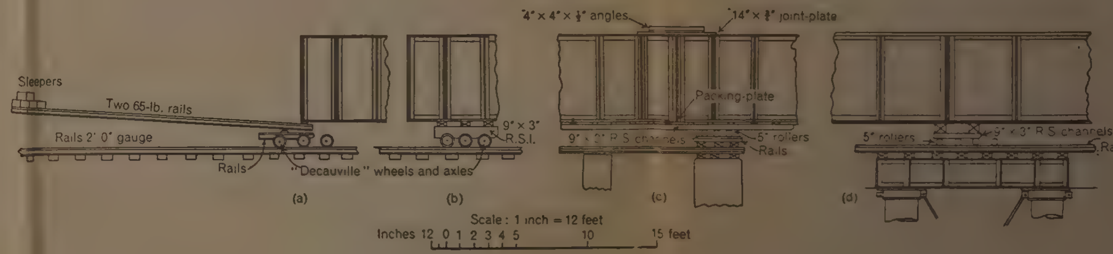
DETAILS OF GRAPNELS, ETC.

Figs: 8.



ROLLING-OUT PLATE-GIRDER SPANS.

Figs: 9.



ARRANGEMENTS FOR ROLLING GIRDERS.

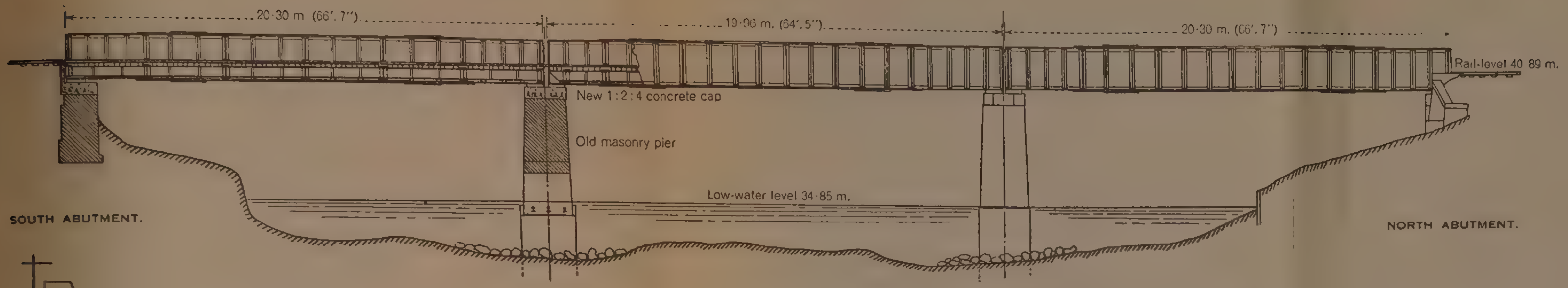
The Institution of Civil Engineers. Journal. April, 1938.

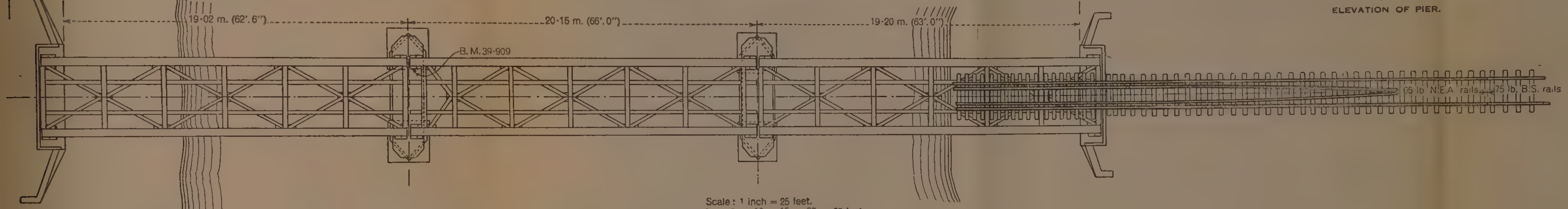
G. C. BLOFIELD.

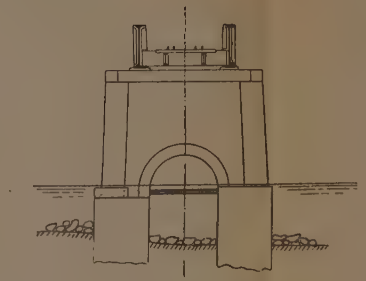
William Clowes & Sons, Limited: London.

THE RECONSTRUCTION OF THE MOCORETÁ AND TIMBOY BRIDGES, ARGENTINE NORTH-EASTERN RAILWAY.

FIGS: 10.







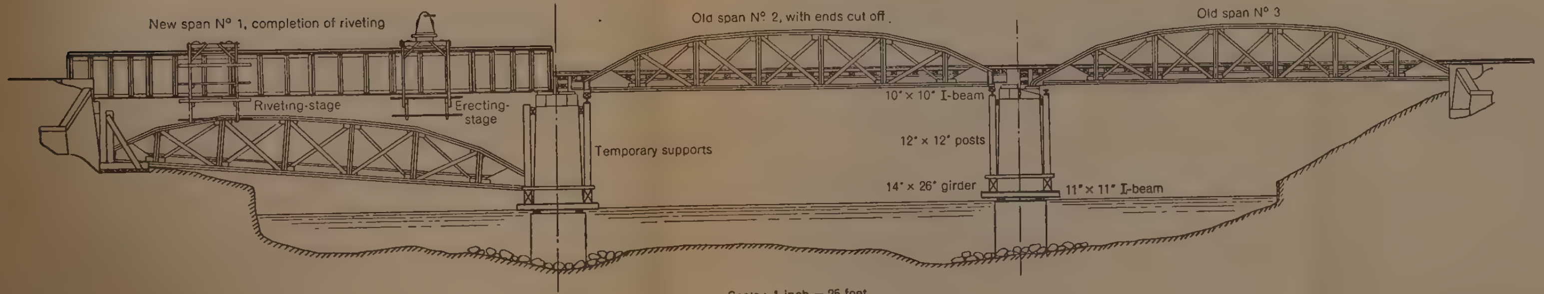
ELEVATION OF PIER.

Scale : 1 inch = 25 feet.
Feet 5 0 5 10 15 20 25 feet

RECONSTRUCTED TIMBOY BRIDGE.

Scale for Figs. 13, 14, 15, and 16.
1 inch = 16 feet.
Feet 5 0 5 10 15 20 feet

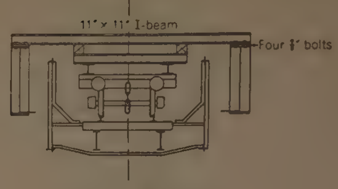
FIG: 11.



Scale : 1 inch = 25 feet.
Feet 5 0 5 10 15 20 25 feet.

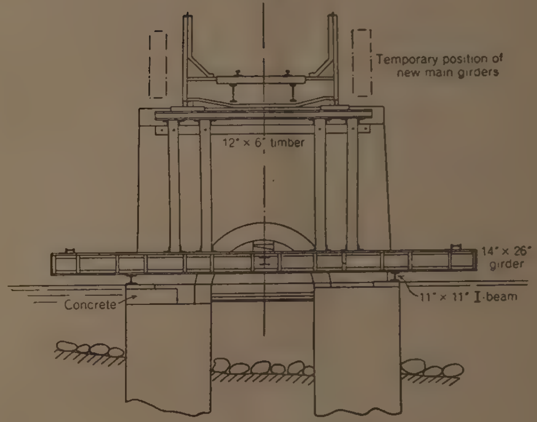
BRIDGE DURING RECONSTRUCTION.

FIG: 13.



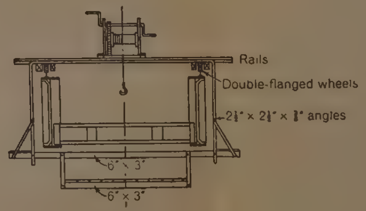
CARRYING-OUT NEW MAIN GIRDERS.

FIG: 14.



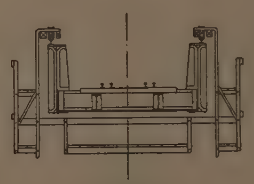
TEMPORARY SUPPORTS AT PIER.

FIG: 15.



STAGE FOR ERECTING FLOOR.

FIG: 16.



RIVETING STAGE.

G. C. BLOFIELD.

WILLIAM CLOWES & SONS, LIMITED : LONDON.

Paper No. 5130.

"Rapid Staining in Granites used in Civil Engineering Work."

By Bernard Howard Knight, D.Sc., Ph.D., M. Inst. C.E., and Rena Gertrude Knight, M.A., M.Sc.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

Introduction.

The attention of the senior Author was drawn to the problem of rapid staining in certain granites used in civil engineering works by reason of trouble which arose with Cornish granite that was sent to London for use in the new Chelsea bridge, which was completed recently. Some of the grey granite rapidly became yellow, even while the dressed blocks were on the site awaiting emplacement in the bridge, and the rapidity with which this took place indicated that the phenomenon was different in degree, if not in kind, from the more familiar normal weathering of rocks. In order to investigate this point, the Authors carried out the research described in this Paper.

The granites concerned came from various quarries in Cornwall, and examination of the dressed stone at the site of Chelsea bridge showed that the staining was not confined to the granite from any one quarry, and that most of the Cornish granites were liable to include types subject to this change. Samples were taken from the bridge-site of both fresh and stained blocks, this being followed by sampling at the quarries of origin, both at the face and from the

[1] Correspondence on this Paper can be accepted until the 15th July, 1938, and will be published in the Institution Journal for October, 1938.—Acting Sec. Inst. C.E.

stock piles. Thin sections of the samples were prepared, and were examined in detail with a petrological microscope.

Studies of the discoloured rock in various stages of staining showed that the rock as freshly quarried had the typical clean grey appearance of Cornish granite; careful examination of the freshly-fractured surfaces with a powerful hand-lens showed, however, that granite liable to subsequent staining is characterized by minute spots of a yellowish colour; these spots are known to the quarry-men as "honey-spots," and are quite a familiar feature to the men.

Preliminary Tests.

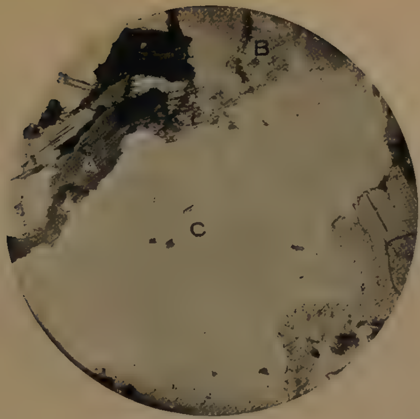
The change in colour of the exterior of the stone from grey to yellow suggested that some iron compound was the source of the trouble, and accordingly the black (biotite) mica in the rock was suspect as being the only bulk mineral containing this element in large amounts. Preliminary support was lent to this view by the application of a simple chemical test. A filter-paper just wetted with dilute sulphuric acid was pressed on to the surface of the stone, and then on to another paper damped with a solution of potassium ferricyanide. A pattern of prussian blue appeared on the paper wherever the yellow stain in the stone had been in contact with the acid paper, this reaction being indicative of the presence of iron.

Petrological Examination of Specimens.

Examination of thin sections of a large range of samples of Cornish granite was then carried out. The types studied included: (*a*) rock from a quarry which has never been known to yield a rock capable of rapid staining; (*b*) unstained rock from a suspect quarry; (*c*) "honey-spot" rock from the same quarry; (*d*) stone which had stained rapidly. It was found by the Authors that the dark mica was the source of the iron staining. It was also found that the mica occurred in two distinct types, distinguishable by optical means in thin section, one type being liable to rapid change, and the other being stable. This and other petrological differences in the rocks studied led to their division into:—type A, unlikely to become stained even after long exposure to the atmosphere; and type B, likely to become stained within a few months after quarrying, and in some cases within a few days.

Type A. Granites of this class contain normal clear quartz, which is virtually free from products of decomposition of other minerals in the minute fissures which are normally present. Felspar is, in general, not markedly decomposed, while the white mica is flaky, and free from pleochroic haloes. Biotite mica has a clean

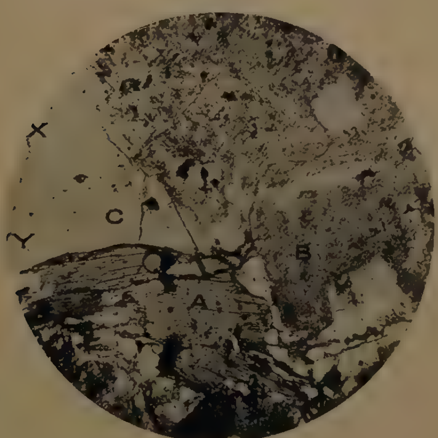
Fig. 1.



×25.

TWO-MICA GRANITE FROM CARNSEW, CORNWALL. STABLE BI-AXIAL BIOTITE CRYSTAL AT A, WITH PLEOCHROIC HALOES, GOOD CLEAVAGES, AND SHARP OUTLINES; SLIGHTLY DECOMPOSED CLOUDY FELSPAR AT B; CLEAR QUARTZ AT C. THIS ROCK WILL NOT STAIN APPRECIABLY UNDER WEATHER-CONDITIONS. (See also *Fig. 2.*)

Fig. 2.

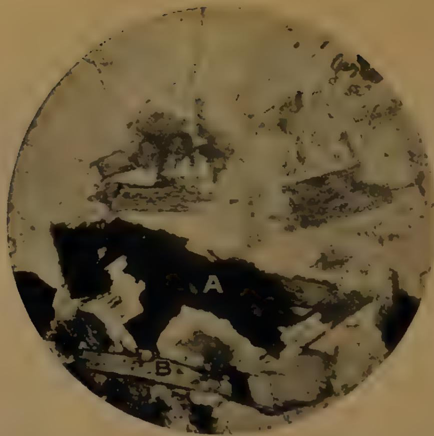


×25.

TWO-MICA GRANITE FROM CREETOWN, KIRKCUDBRIGHTSHIRE. UNSTABLE UNI-AXIAL BIOTITE MICA AT A, BREAKING DOWN AND STAINING IN DECOMPOSED FELSPAR B, AND IN FISSURED QUARTZ C. THE ACTION IS PROGRESSIVE, AND WILL ULTIMATELY STAIN THE WHOLE SURFACE OF THE STONE.

5130.

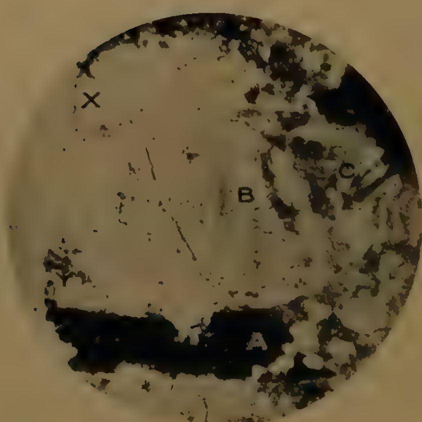
Fig. 3.



× 25.

TWO-MICA GRANITE FROM HIGHER SPARGO, CORNWALL. UNSTABLE UNI-AXIAL BIOTITE MICA AT A, IN THE SAME SECTION AS STABLE BI-AXIAL BIOTITE MICA B.

Fig. 4.



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TWO-MICA GRANITE FROM PELASTINE, CORNWALL. WEATHERED EDGE AT XY, WITH UNSTABLE UNI-AXIAL BIOTITE AT A, BEGINNING TO BREAK DOWN NEAR THE SURFACE AND TO STAIN FISSURED QUARTZ AT B, THE STAINING HAVING NOT YET REACHED THE FELSPAR C.

appearance, with sharp edges, and cleavages of a well-marked character (*Fig. 1*). The pleochroism (that is, the change of colour during rotation in plane-polarized light) is very marked, the range being from almost colourless to a deep red of a crimson tinge. The interference-figure obtained in convergent polarized light is bi-axial, with a negative sign, the optical axial angle being small. Very little alteration of the biotite has occurred, but some chloritic material is sometimes present. Inclusions are relatively few, and are surrounded by markedly pleochroic haloes.

Type B. The quartz in this type of granite is blebby, and the minute fissures are either wholly or partially filled with strings of opaque iron oxide, and occasionally with yellow isotropic material which passes into scaly colourless micaceous alteration-products nearer the biotite crystals. In the more extreme cases, the fissures extend into the felspars, which are themselves frequently badly decomposed. In addition to the scaly micaceous material mentioned, and the normal white mica as found in type A, a third kind of white mica is found. This contains pleochroic haloes, and itself appears sometimes to be faintly yellow in colour. Some of these flakes contain patches of dark brown mica which pass imperceptibly into the white mica without any sharp line of demarcation, such as would be reasonably expected to occur if the dark mica were true inclusions in the light mica. The inference from these facts is that the third type of white mica is an alteration-product after dark mica.

The dark mica in this rock is of two types: (i) similar to that already described as occurring in type A granites; and (ii) of a kind confined to the type B granites, and considered by the Authors to be the mineral responsible for the rapid surface-staining. This second type of dark mica is of a deep red-brown colour, and is pleochroic, from brown-red to vermilion-red. Its appearance is amorphous, patchy or blotchy, suggesting that the mineral is not so completely crystalline as the other type of biotite mica. The cleavages are not well marked, and the edges are ill-defined (*Figs. 2, 3* and *4*), being frequently bordered by a green micaceous or chloritic product which sometimes passes into scaly white micaceous material. The interference-figure is either completely uniaxial or nearly so (pseudo-uniaxial). Inclusions are commoner and larger than in the other type of biotite; they are opaque and non-pleochroic. As mentioned above, strings of decomposition-products extend from this type of biotite into the adjacent quartz and felspar, and in some cases these minerals appear to be stained faintly yellow, even in thin section. The results of the petrological examination are shown in Table I (pp. 548 and 549).

TABLE I.—RESULTS OF

Locality.	Rock-type.	Quartz.	Felspar.	Biotite		
				Appearance.	Cleavage.	Haloes.
Swell Tor, Devon.	Two-mica granite.	Fissured, normal.	Rather decomposed, fissured.	Sharp.	Good.	Pleochroic.
Carbilly, Cornwall.	,,	,,	Fairly fresh.	Clear.	Marked.	,,
De Lank, Cornwall.	,,	Blebby, shows staining.	Rather decomposed.	Amorphous.	Poor.	Non-pleochroic.
Kit Hill, Cornwall.	,,	Normal.	Average decomposition.	Indistinct, blotchy.	,,	,,
Tresahor, Cornwall.	,,	,, stained.	,, stained.	Ragged edges, amorphous.	,,	,,
Higher Spargo, Cornwall.	,,	,,	,,	,,	,,	,,
Carnsew, Cornwall.	,,	Some normal; some blebby, stained.	,,	,,	,,	,,
Pelastine, Cornwall.	,,	,,	,,	,,	,,	,,
Trevolvis, Cornwall.	,,	Normal.	,,	,,	,,	,,
Penzance, Cornwall.	,,	Fissured, no staining.	Decomposed.	Clean.	Good.	Pleochroic.
Creetown, Kirkcudbright.	,,	Fissured, blebby.	Kernel moderate decomposition.	Amorphous.	Poor.	Few; non-pleochroic.
Fyfe, Norway.	,,	Blebby, small fissuring.	Moderate decomposition.	Amorphous.	Poor.	Non-pleochroic.
Cairn, Norway.	Biotite granite.	Normal, little fissuring.	Very fresh.	Good.	Good.	Pleochroic.

CAUSES OF RAPID STAINING.

The data given above were sufficient to confirm the Authors in their opinion that an unstable form of biotite mica was the primary cause of the rapid staining of the stone, and their theory was tested by examination of other granites which were known to become discoloured in a short time on exposure to the atmosphere. These

PETROLOGICAL EXAMINATION.

mica.			Horn-blende.	Augite.	Remarks.
Pleo-chroism.	Colour.	Figure.			
Strong.	Brown, slight red tinge.	Bi-axial.	—	—	Fresh.
,,	Brown to crimson.	,,	—	—	,,
Poor.	Almost blood-red.	Uni-axial.	—	—	Liable.
,,	Ver-milion.	,,	—	—	,,
,,	,,	,,	—	—	,,
,,	,,	,,	—	—	,,
,,	,,	,,	—	—	,,
,,	,,	,,	—	—	,,
,,	,,	,,	—	—	,,
Good.	Pale brown.	Bi-axial.	—	—	From 100-year-old building; fresh.
Poor.	Brown-red.	Uni-axial.	—	—	Stained with limonitic decomposition-products.
Poor.	Dirty brown.	Uni-axial.	—	—	Liable.
Strong.	Brown.	Not found.	—	—	Fresh.

were granites from Kirkcudbrightshire, and one from Norway. In both of these granites, biotite mica was present as well as white mica, the former exhibiting the characteristic blotchy, amorphous appearance and pseudo-uni-axial interference-figure found in the unstable biotite of the Cornish granites.

It should be stressed that in no case was the whole of a quarry found to yield "honey-spotted" rock, and that careful selection of

material will enable firms to supply a fresh grey granite which will retain its original colour. It should also be mentioned that the staining is merely a surface change affecting the outer skin only of the rock, which detracts in no way from the structural strength of the stone, and is only of importance if the engineer requires the completed structure to be of a uniform grey colour.

The work of the Authors goes to show that any two-mica granite containing the unstable type of biotite mica is liable to show a yellow surface-stain on exposure to the atmosphere, and that in general, the greater the amount of that mineral, the more rapid the rate of change of colour.

Reference to geological literature shows that in Dartmoor Dr. Alfred Brammall [1] has found that " . . . in the more acid granites, muscovite is common, and the associated biotite . . . often shows partial and blotchy discolouration. The occasional occurrence of pleochroic haloes in pale yellow to almost colourless flakes suggests that the latter may approximate to phengite or cryophyllite evolved by reaction processes from the more basic species. . . ." Further, in commenting on the chemical composition of five analysed biotites, Dr. Brammall remarks " . . . interpreting the variation broadly, with (a) increasing silica percentage, (b) progress from basic to acid, and older to newer granites, the micas change in composition away from phlogopite towards iron-rich lepidomelane . . . this suggests the familiar reaction-process in which the MgO/FeO ratio is progressively reduced in favour of FeO as silica increases." These observations support the opinion of the Authors that the peculiar biotite mica occurring in some of the Cornish granites of an acid type (that is, those containing muscovite) was unusually rich in iron, and sufficiently unstable to liberate some of this iron on exposure to the atmosphere. This point was accordingly discussed with Dr. Brammall, who agreed that the Authors' explanation was perfectly feasible. He also supplied them with a written communication relating to one of the granites investigated by them, and kindly gave permission for quotation of the following therefrom :—" On the evidence of thin sections alone, I concluded that the mischief was due to the hydrolysis of the peculiar reddish-brown, iron-rich, unstable biotite variety characteristic of the more acid granites in the Cornwall-Devon province. This basic (lepidomelanic ?) mica is first hydrolysed to a yellowish-green chloritic substance rarely specific in its properties, but apparently containing FeO in some feeble, vulnerable combination. I still entertain the belief that this chloritic matter

[1] " The Dartmoor Granite." Quarterly Journal Geol. Soc., vol. 88 (1932), p. 183.

is essentially a hydrosol until exposed to the atmosphere or to circulating ground-water. It is next oxidized; it usually yields a plexus or mat of flakes (sericitic or kaolinitic) in which the ferric hydrate (limonitic) is mordanted. The ferric hydrate has a very notable degree of "spreading power," and is mordanted also by the absorbent fabric of destabilized felspars. Some of the stronger blades of bleached biotite are often margined by this limonitic matter, which also penetrates along the mica-cleavages. Its occurrence as a canal-like network in the fractures of quartz, and the cleavages and fractures of fresh felspar, is common. A check-test on fresh grey and stained granite from the same block showed a marked increase in total water. From these facts, I concluded that the iron-stain could be reasonably attributed to the destabilization of the biotite." It was also suggested that the presence of iron pyrites might be a contributory factor, but this was not borne out by earlier work of the senior Author.[1]

Artificial Acceleration of Staining.

In addition to the petrological studies described, the Authors carried out laboratory tests designed to accelerate the change. Since the latter appeared to be one of oxidation, concentrated nitric acid was chosen as the reagent. Freshly-fractured hand-specimens of granite known to be liable to stain were treated with a few drops of acid on half of an exposed face, the other half being left untreated for purposes of comparison. It was found that in a few days there was a marked difference in the colour of the treated and untreated surfaces, and that this only occurred with stone liable to the change. It was also found that the difference was less marked in those granites which were known to be less liable to staining. On the other hand it was found that a test-period of 7 days was not sufficient to detect with absolute certainty all granites shown by petrological methods to be liable to the trouble.

Results of Investigation.

The results of the investigation may be summarized as follows:—

(*a*) Granites which are liable to stain on exposure to the atmosphere contain both white and dark mica.

(*b*) The staining is due to the presence of an unstable dark mica, which can be recognized in thin section by its

[1] "The Influence of Weather on Granite Kerbs, Setts and Broken Stone Roads." Quarry and Surveyors' and Contractors' Journal, vol. 29 (1924), p. 186.

distinctive optical properties: it is restricted and local in its occurrence, even in the two-mica granites.

(*c*) The change of colour is a surface phenomenon only, and is of no structural significance.

(*d*) The staining may be artificially accelerated by the use of nitric acid, but the behaviour of the stone under this test is to be taken as indicative only, and not as conclusive of its behaviour when exposed for long periods to the atmosphere.

(*e*) Finally, it was found that the stain can be removed by washing with dilute sulphuric acid, but that the effect of this treatment is only temporary, and that the trouble will reappear.

The Paper is accompanied by eight photographs, from some of which the half-tone page-plate has been prepared.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Joint Sub-Committee on Vibrated Concrete.

THE First Interim Report of the Joint Sub-Committee with the Institution of Structural Engineers appointed to consider the effect of vibration on concrete appeared in the Institution Journal for March, 1937.[1] In that Report were given the results of tests designed to show the effect on the strength and density of the concrete, of variations in the water/cement ratio, frequency of vibration, duration of vibration, and maximum acceleration, one variable being studied at a time.

It was indicated that further experimental work was required, and, as a result of the response to an appeal for further financial support, it has been possible to complete the investigation [2] carried out with the special vibration machine constructed for the tests. The results and conclusions to be drawn from this further research have been embodied in the Second Interim Report which is given below.

There still remain to be investigated the practical difficulties involved in the use of vibration as a method of compacting. Before work in this direction can usefully be started, however, it is necessary to be able to measure, on the work, the degree and extent of vibration in concrete during compacting, and work is now in progress on the development of a simple, compact and inexpensive instrument which, when placed in concrete, will indicate the characteristics of the vibration of the concrete at the point of measurement.

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[1] Journal Inst. C.E., vol. 5 (1936–37), p. 435. (March 1937.)

[2] The investigation has been carried out by Dr. F. G. Thomas, Assoc. M. Inst. C.E., at the Building Research Station as work for the Building Research Board in co-operation with the Joint Sub-Committee.

Investigation on the Vibration of Concrete. Interim Report No. 2.

Further Tests on Concrete Compacted on a Vibrating Table.

Introduction.

A study of the effect of compacting concrete by vibration on the physical properties of the concrete is being made as part of the general research programme at the Building Research Station. Since April, 1936, the work has been carried out in co-operation with a Joint Sub-Committee of the Institution of Civil Engineers and the Institution of Structural Engineers.

In Interim Report No. 1, published in March, 1937,[1] the effects of the time, acceleration and frequency of vibration were considered in relation to the efficacy of compaction of a 1-to-6 mix (by weight) of concrete with a particular grading of river sand and gravel.

The work has now been extended to cover other cement-contents from a 1-to-3 mix to a 1-to-9 mix (by weight) and to determine the effect of varying the grading of the aggregate. Besides strength tests, the work has included brief studies of the creep, shrinkage, and extensibility of some vibrated concretes and of the bond between concrete and its reinforcement.

General Details of Tests.

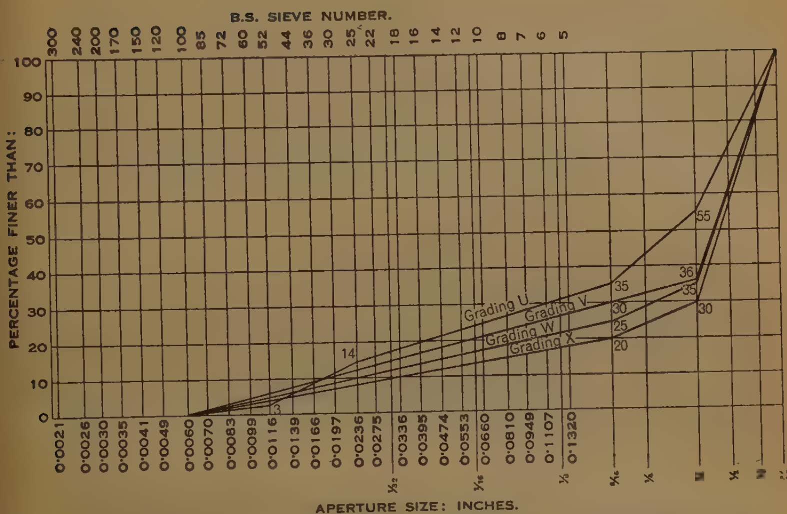
All specimens were cast on the larger vibrating table described in the previous Report. In all cases the frequency of vibration was 3,000 per minute. Ordinary Portland cement was used throughout with river sand and gravel. The maximum size of aggregate was ¾ inch and the gradings investigated are shown in *Figs. 1* to *3*. Specimens for strength tests were cast in 4-inch cubes in batches of three, and two similar batches were made for each variation of mix or method of vibration. The difference between the average strengths of two such batches was never greater than 12 per cent., and was usually less than 5 per cent.

Tests Carried Out.

The further work to be described and discussed in the present Report may be summarized as follows :—

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 436. (March, 1937.)
Journal Inst. Struct. E., vol. 15 (New Series, 1937), p. 133.

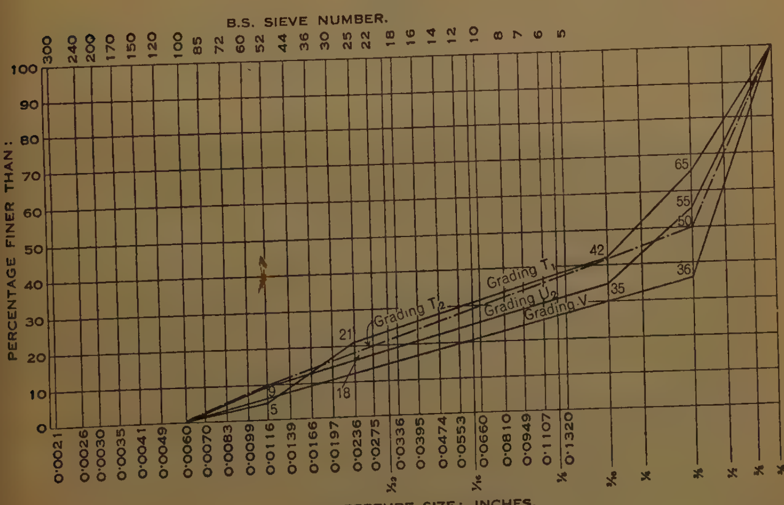
Fig. 1.



GRADINGS FOR VIBRATION COMPACTING TESTS: 1-TO-6 MIXES AND RICHER.

(1) The relationship between the water/cement ratio and strength of vibrated concrete has been determined for a series of concretes with different cement-contents. For each cement-content the effect

Fig. 2.

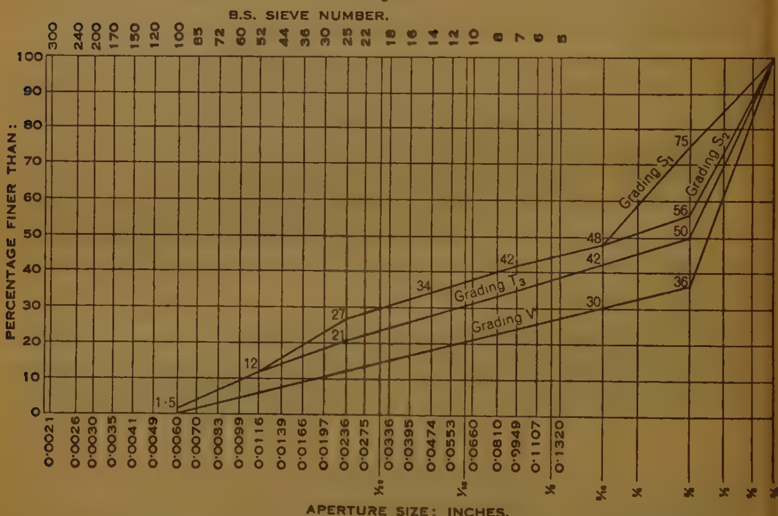


GRADINGS FOR VIBRATION COMPACTING TESTS: 1-TO-$7\frac{1}{2}$ MIXES.

of varying the grading of the aggregate was investigated (*a*) for a range of times of vibration of from 15 seconds to 6 minutes with a constant acceleration of 4*g*; and (*b*) for a range of accelerations of from 1*g* to 10*g*, with a constant time of 2 minutes.

(2) The effect of prolonged vibration of concrete on its strength has been determined with a 1-to-6 mix with a water/cement ratio of 0·60 (by weight). The effect of intermittent vibration has also been examined.

Fig. 3.



GRADINGS FOR VIBRATION COMPACTING TESTS: 1-TO-9 MIXES.

(3) Measurements have been made of the shrinkage, elastic and creep properties of a 1-to-6 mix with various water/cement ratios ranging from 0·35 to 0·60 (by weight).

(4) The bond between concrete and steel has been determined with a 1-to-6 mix with water/cement ratios of from 0·35 to 0·60 (by weight). These tests included the cases (*a*) with the bar vertical and (*b*) with the bar horizontal, during placing of the concrete. The effect of subsequent vibration on the bond-strength was also investigated.

(5) The modulus of rupture was obtained for a 1-to-6 mix with water/cement ratios of from 0·35 to 0·60 (by weight).

For the tests under (2)–(5) above the acceleration was 4*g*, and the time of vibration was adjusted in each case to give satisfactory consolidation.

(1) *Strength of Vibrated Concrete, and its Relationship to Cement- and Water-Content and the Grading of the Aggregate.*

1-to-6 mix.—The gradings used for the 1-to-6 mix are shown in *Fig. 1* (p. 555). In the previous Report, tests were described for this mix and grading V. It was thought possible that with the lower water-contents possible with vibration compacting, the sand-content could with advantage be reduced, and two of the gradings (W and X) were arranged on this basis. A third grading (U) with a higher sand-content was introduced as a possible limit beyond which the amount of water required for satisfactory consolidation might appreciable reduce the usefulness of vibration.

The results of strength tests showing the effect of varying the time of vibration or the acceleration are included in Tables I (p. 558) and II (p. 559) for a water/cement ratio of 0·40. This was found from preliminary tests to be the lowest water-content consistent with good compaction of a 1-to-6 mix, for the conditions of vibration used.

For times of vibration greater than 2 minutes with an acceleration of $4g$, there was little difference between concretes of gradings U, V, and W. For shorter periods of vibration the use of grading W leads to lower strengths than grading U or V. For all periods of vibration, grading X resulted in less satisfactory consolidation than that obtained with the other gradings. The strength with this grading was reduced, and the surface-appearance was very poor indeed, with considerable honeycombing.

When the time of vibration was fixed at 2 minutes and the acceleration varied it was found (see Table II) that above accelerations of $4g$ there was little difference between the degree of consolidation and strength of concretes with gradings U, V, and W. Below this acceleration, grading W is not so satisfactory as gradings U and V. Grading X gave poor results throughout.

The surface-appearance with the highest sand-content (grading U, 35 per cent. sand) was poor compared with that with 30 per cent. sand (grading V), so that it appears that a further increase in sand-content beyond 35 per cent. would lead to very poor consolidation and reduced strengths.

Mixes Richer than 1 to 6.—Similar tests to those described above for a 1-to-6 mix have been made with mixes of 1-to-4½ and 1-to-3 (by weight). In these cases, however, owing to the small differences obtained in a 1-to-6 mix with gradings U, V, and W, only the extreme gradings U and X were investigated. The water/cement ratios chosen were 0·375 (by weight) for the 1-to-4½ mix and 0·30 for the 1-to-3 mix. The results are included in Tables I and II, from which it will be seen that the differences between the two gradings have no important influence on the strengths of the concretes for these mixes.

TABLE I.—EFFECT OF TIME OF VIBRATION ON STRENGTH OF CONCRETE.

(Frequency 3,000 per minute. Acceleration 4*g*.)

Mix (by weight).	Water/cement ratio (by weight).	Grading.	7-day strength of 4-inch cubes for various times of vibration : lb. per square inch.							
			15 seconds.	30 seconds.	45 seconds.	1 minute.	1½ minute.	2 minutes.	3 minutes.	6 minutes.
1-to-3	0·30	U	6,580	7,300		7,740				7,780
		X	5,930	7,260		7,420				7,420
1-to-3	0·275	U				7,760		8,120		
		X				8,100		8,640		
1-to-4½	0·375	U	6,070	6,500		6,450				6,650
		X	5,150	6,080		6,540				6,820
1-to-6	0·40	U		4,240		5,675			6,925	7,150
		V		4,010		5,410			6,270	6,410
		W		3,570		4,830			7,015	7,150
		X		3,150		4,280			4,610	6,020
1-to-7½	0·50	T_1			3,630		4,260		4,400	4,340
		T_2			4,350		4,420		4,600	4,210
1-to-9	0·60	T_3	2,400			3,210		3,370		3,300
		S_1	1,200			2,100		2,260		2,900

In view of the fact that a reduction in water/cement ratio is often possible when the sand-content is reduced, a few tests were made with a 1-to-3 mix with a water/cement ratio of only 0·275, but there was still no significant difference between the results with the two gradings.

Mixes Leaner than 1-to-6.—In view of the high strengths that are obtainable with a 1-to-6 mix when a low water-content is used in conjunction with vibration compacting, it was decided to investigate to what extent it is possible to reduce the cement-content in vibrated concrete and still to secure adequate strength for particular jobs. Tests have therefore been made to investigate the strengths of 1-to-7½ and 1-to-9 mixes with various gradings.

TABLE II.—EFFECT OF ACCELERATION ON STRENGTH OF VIBRATED CONCRETE.

(Frequency 3,000 per minute. Time of vibration 2 minutes.)

Mix (by weight).	Water/cement ratio (by weight).	Grading.	7-day strength of 4-inch cubes for various accelerations: lb. per square inch.							
			1g.	1½g.	2g.	3g.	4g.	5g.	7g.	10g.
1-to-3	0·30	U	3,920		8,120			8,310		8,300
		X	3,450		7,580			7,660		7,670
1-to-4½	0·375	U	2,470		5,700		6,620			6,870
		X	2,580		5,540		6,950			6,960
1-to-6	0·40	U			5,200	6,420		6,540	6,600	
		V			4,600	6,740		7,060	7,100	
		W			3,400	5,500		6,800	6,900	
		X			2,600	3,700		5,000	5,830	
1-to-7½	0·50	T_1		1,310		4,200		4,480		4,230
		T_2		2,270		4,550		4,840		5,020
1-to-9	0·60	T_3	600		3,100		3,370			3,400
		S_1	300		900		2,260			2,160

(*a*) *1-to-7½ mix.*—Preliminary tests with the four gradings given in *Fig. 1* (p. 555) and a 1-to-7½ mix yielded such poor results that it was decided to try other gradings, with higher sand-contents with slightly more fine particles. These are shown in *Fig. 2* (p. 555), in which grading V is included for comparison. In initial tests to determine the most useful water-content it was found that there was little difference between the concretes using gradings U_2 and T_2, so that it was decided to investigate fully only gradings T_1 and T_2. The water/cement ratio chosen was 0·50. From *Fig. 2* it will be noticed that the sand-content was the same for the two gradings. With T_2 the material passing a No. 52 sieve was 9 per cent., as against 5 per cent. with T_1; and the material between $\frac{3}{16}$ inch and $\frac{3}{8}$ inch was decidedly less with T_2.

The results (see Tables I and II) show that the use of grading T_2 leads to better consolidation, particularly when a high acceleration is used.

(*b*) *1-to-9 mix.*—Two main gradings, S and T_3, were investigated for a 1-to-9 mix, as shown in *Fig. 3* (p. 556), grading V again being included for comparison. A water/cement ratio of 0·60 was found to be necessary to obtain well-compacted cubes with good surface-appearance. When the time of vibration was 2 minutes an acceleration of $2g$ was sufficient to compact the concrete with grading T_3, whereas satisfactory consolidation was not obtained with grading S even at an acceleration of $10g$.

When the time of vibration was increased, with a constant acceleration of $4g$, the difference between the strengths with the two gradings became less marked, but it appeared that grading S was quite unsuitable for use with a 1-to-9 mix of vibrated concrete.

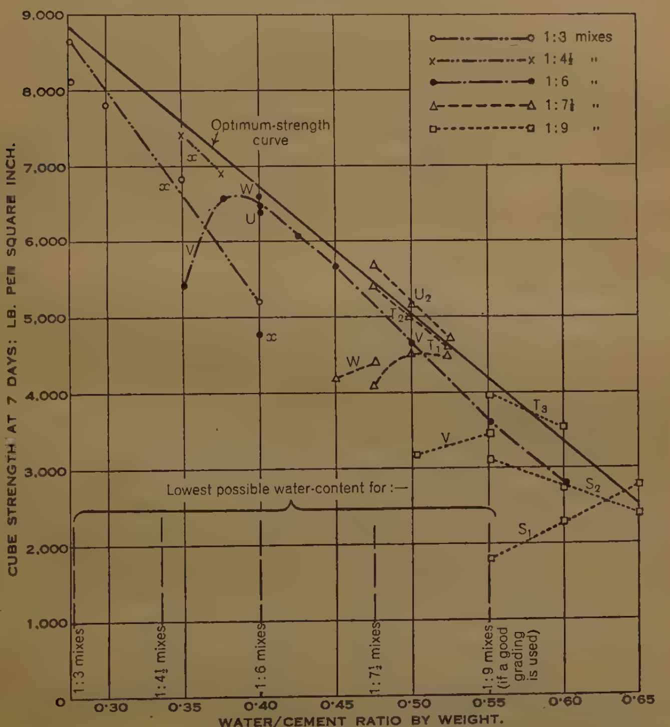
The effect of reducing the material between $\frac{3}{16}$ inch and $\frac{3}{8}$ inch was investigated by a few tests with grading S_2, shown in *Fig. 3* (p. 556). With a water/cement ratio of 0·60 the strength of concrete with aggregate grading S_2 was 1·23 times that obtained with grading S_1. When the water/cement ratio was reduced to 0·55 the ratio was 1·74. It is therefore clear that the grading with a 1-to-9 mix is very important.

Relationship between Water/Cement Ratio and Concrete-Strength.—The results of tests on all proportions and gradings for an acceleration of $4g$ and a time of vibration of 2 minutes are shown in *Fig. 4*. Here a curve has been drawn to indicate the maximum strengths which can be obtained by vibration for water/cement ratios between 0·275 and 0·65. This curve is for all practical purposes a straight line between the limits investigated. This figure shows the considerable variation in strength with lean mixes using different gradings. It is interesting to note that for the 1-to-3 and 1-to-6 mixes an increase in water/cement ratio beyond the value giving greatest strength causes a greater reduction in strength than that indicated by the straight line representing the best conditions. That is, for a particular water/cement ratio a higher strength may possibly be obtained with, say, a 1-to-6 mix than with a 1-to-3 mix, although both mixes can be compacted to the same density-ratio. For example, the strength of a 1-to-3–0·40 mix was 20 per cent. less than that of a 1-to-6–0·40 mix. With a water/cement ratio of 0·35, on the other hand, it was not possible to get satisfactory consolidation with a 1-to-6 mix and more cement would have to be used. The lowest water-contents which could be successfully used for the various mixes in the present tests are shown in *Fig. 4*.

(2) *Effect of Prolonged and Intermittent Vibration on the Strength of Vibrated Concrete.*

(*a*) *Effect of Prolonged Vibration during Setting and Hardening.*—In the previous Report it was pointed out that if the period of vibration were increased considerably, the 7-day strength of the concrete

Fig. 4.



Relationship Between Strength and Water/Cement Ratio of Vibrated Concrete.

might be appreciably affected, particularly with wet mixes. With a 1-to-6 mix with a water/cement ratio of 0·40 the effect was not important, but with a water/cement ratio of 0·60, a 70-per-cent. increase of strength was obtained when the period of vibration was extended from 2 minutes to 3 hours.

Subsequent tests have shown that the increase in strength with wet mixes is the result of a decrease in water-content of the main body of the concrete. The vibration with a wet mix brings a considerable amount of water to the surface and this is allowed to run off so

that, for example, with a 1-to-6 mix with a nominal water/cement ratio of 0·60, the actual water/cement ratio is only about 0·40 after 3 hours' vibration. The strengths at 7 days of such a mix vibrated for various periods are given in Table III, together with the

TABLE III.—THE EFFECT OF PROLONGED VIBRATION ON THE STRENGTH OF VIBRATED CONCRETE.

Mix (by weight).	Grading (by weight).	Period of vibration	Water/cement ratio at end of vibration.	Strength corresponding to water/cement ratio: lb. per square inch.	Actual strength obtained: lb. per square inch.
1-to-6	V	2 minutes	0·58	2,900	2,950
		1½ hour	0·42	6,300	4,300
		3 hours	0·40	6,550	5,040
		6 hours	0·40	6,600	5,200

water/cement ratios at the end of the period of vibration, as determined analytically. In this Table are included the strengths that could have been obtained at 7 days by vibrating a 1-to-6 mix for only 2 minutes with a water/cement ratio corresponding to the value at the end of each prolonged vibration. It is clear that the results for prolonged vibration are reasonably consistent with the assumption that the effect is merely to cause a reduction in water/cement ratio.

(b) *Effect of Intermittent Vibration.*—It was thought that if a concrete mix were compacted by vibration, allowed to remain undisturbed for some time and then subjected again to vibration (due possibly to the compacting of subsequently-placed concrete) there might be a detrimental effect on the strength of the concrete. To investigate this a series of tests was made in which the concrete was vibrated for 3 minutes and then allowed to remain undisturbed for various periods up to 17 hours before being vibrated again for a further period of 3 minutes. The results, given in Table IV, show that the subsequent vibration had no marked effect on the concrete strength.

(3) *Shrinkage, Elasticity, and Creep of Vibrated Concrete.*

Measurements have been made of the shrinkage, elasticity, and creep of a 1-to-6 mix, (a) when compacted by hand with a water/cement ratio of 0·60, and (b) when compacted by vibration with water/cement ratios ranging from 0·60 to 0·35.

The shrinkage measurements were made on beams 36 inches long with a 4-inch-square cross-section. The elasticity measurements were made on cylinders 10 inches long and 4 inches in diameter, loaded in compression at ages of 7 and 28 days. The creep measure-

TABLE IV.—THE EFFECT OF RE-VIBRATING CONCRETE ON ITS STRENGTH.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	7-day strength of concrete: lb. per square inch.						
			Vibrated 3 minutes.	Vibrated 3 minutes and re-vibrated 3 minutes after an interval of:—					
				15 minutes.	30 minutes.	1 hour.	2 hours.	5 hours.	17 hours.
1-to-6	0·60	A	2,710	3,020	2,590	2,950	3,200	3,030	2,870
1-to-6	0·50	A	4,640	4,785	4,470	4,820	4,960	4,570	4,585
1-to-6	0·40	A	7,020	7,265	7,210	7,580	7,720	7,790	7,220

ments were made on cylinders 10 inches long and 4 inches in diameter, loaded at an age of 3 days to a stress of 715 lb. per square inch (the total load on the 4-inch diameter specimen being 4 tons). The results are given in Tables V to VII (p. 564), from which it will be seen that the use of a drier mix with vibration considerably reduces both the shrinkage and the creep of the concrete, and increases the modulus of elasticity.

(*4*) *Bond between Vibrated Concrete and Steel.*

A series of tests has been made to determine the bond-strength between steel and concrete using the same mixes as in the creep, shrinkage, and extensibility tests. A ¾-inch diameter mild-steel bar was embedded centrally in a 4-inch cube of concrete, either (*a*) with the bar vertical or (*b*) with the bar horizontal when the concrete was placed. At an age of 7 days the bars were pulled from the concrete and measurements were made of the slip at the free end of the bar. Typical results are shown in *Fig. 5* (p. 565) and further data are given in Table VIII (p. 566). It is clear from these that the bond-strength is considerably improved by using a dry mix with vibration compacting. It is interesting to note, however, that the use of vibration with a wet mix may cause a decrease in bond-strength, particularly for horizontal reinforcement.

Some specimens were re-vibrated 18 hours after casting, in order to

TABLE V.—SHRINKAGE OF VIBRATED CONCRETE.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	Method of compacting.	Shrinkage per unit length * in units of 10^{-6}, at age:				
				7 days.	14 days.	28 days.	3 months.	6 months.
1-to-6	0·60	V	Hand-rammed	54	85	123	219	263
	0·60		Vibrated †	48	73	88	181	225
	0·50		,,	25	46	72	133	174
	0·40		,,	27	50	77	125	160
	0·35		,,	23	42	64	113	138

* Measurements started when 1 day old.
† Vibrated at 3,000 vibrations per minute and 4*g* until well compacted.

TABLE VI.—ELASTICITY OF VIBRATED CONCRETE.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	Method of compacting.	Modulus of elasticity,* in units of 10^6 lb. per square inch, at age:	
				7 days.	28 days.
1-to-6	0·60	V	Hand-rammed	4·3	4·7
	0·60		Vibrated *	4·3	4·8
	0·50		,,	4·9	5·4
	0·40		,,	6·5	6·8
	0·35		,,	6·9	7·1

* Vibrated at 3,000 vibrations per minute and 4*g* until well compacted.

TABLE VII.—CREEP OF VIBRATED CONCRETE.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	Method of compacting.	Creep per unit length,* per lb. per square inch, in units of 10^{-8}, at age:				
				7 days.	14 days.	28 days.	3 months.	6 months.
1-to-6	0·60	V	Hand-rammed	14	24	32	43	52
	0·60		Vibrated †	15	23	30	42	48
	0·50		,,	6	10	14	20	23
	0·40		,,	3	6	9	11	12
	0·35		,,	4	7	9	12	14

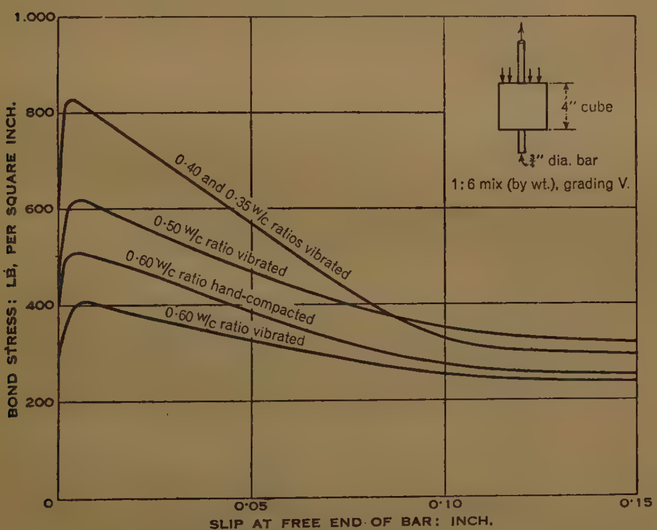
* Concrete loaded at age of 3 days to a stress of 715 lb. per square inch (total load 4 tons).
† Vibrated at 3,000 vibrations per minute and 4*g* until well compacted.

determine whether such vibration would affect the bond-strength. The results, included in Table VIII (p. 566), show, however, no great difference from those obtained with the other specimens.

(5) *Modulus of Rupture.*

The modulus of rupture of a 1-to-6 mix with water/cement ratios of from 0·35 to 0·60 (by weight) was determined from transverse tests on 16-inch by 4-inch by 4-inch prisms. The results are given in Table IX (p. 567). It will be seen that the increase in the modulus of rupture when a dry mix is used is not so great proportionately as the increase in compressive strength.

Fig. 5.



BOND-STRENGTH OF VIBRATED CONCRETE.

(6) *Extensibility.*

A few tests have been made to determine the extensibility (or ability to stretch without cracking) of vibrated concrete; these indicated that the extensibility is not less than that of hand-compacted concrete.

CONCLUSIONS.

It should be clearly realized that the conclusions given hereunder refer only to the particular concrete used in the tests, and that they

TABLE VIII.—BOND-STRENGTH OF VIBRATED CONCRETE.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	Treatment.	Vertically reinforced.			Horizontally reinforced.		
				Bond stress at initial slip : lb. per square inch.	Maximum bond stress : lb. per square inch.	Slip at maximum bond stress, in units of 10^{-4} inch.	Bond stress at initial slip : lb. per square inch.	Maximum bond stress : lb. per square inch.	Slip at maximum bond stress, in units of 10^{-4} inch.
1-to-6	0·60	V	Hand-compacted	375	510	25	162	251	65
			Vibrated *	288	418	50	51	129	107
	0·60		Re-vibrated †	333	442	40	66	146	80
			Vibrated	418	620	45	220	350	50
	0·50		Re-vibrated †	353	596	45	227	306	40
			Vibrated	556	841	33	325	584	60
	0·40		Re-vibrated †	536	770	32	320	577	55
			Vibrated	658	820	20	423	600	45
	0·35		Re-vibrated †	677	811	17	449	700	45

* Vibrated at 3,000 vibrations per minute and $4g$ until well compacted.

† The re-vibrated specimens were vibrated again for a period of 2 minutes, 18 hours after the first vibration.

TABLE IX.—MODULUS OF RUPTURE OF VIBRATED CONCRETE.

Mix (by weight).	Water/cement ratio (by weight).	Grading.	Method of compacting.	Modulus of rupture : lb. per square inch at age	
				7 days.	28 days.
1-to-6	0·60	V	Hand-rammed	400	460
	0·60		Vibrated *	370	460
	0·50		,,	470	530
	0·40		,,	500	550
	0·35		,,	530	650

* Vibrated at 3,000 vibrations per minute and $4g$ until well compacted.

may require revision for larger masses of concrete and different types of vibration.

(1) The grading of the aggregate does not affect the strength of vibrated concrete, with a particular water/cement ratio, when the vibration is sufficient for satisfactory consolidation of the concrete. The grading is of less importance for mixes richer than 1-to-6 (by weight) than for leaner mixes. The use of a 1-to-9 mix requires careful consideration and tests to determine a suitable grading.

(2) For mix proportions of from 1-to-3 to 1-to-9 (by weight) and water/cement ratios of between 0·275 and 0·65, the relationship between water/cement ratio and the optimum strength obtainable with the present type of vibration is practically linear.

(3) In some cases the use of a very rich mix with a particular water/cement ratio may lead to a lower strength than a leaner mix with the same water/cement ratio. This applies when the water-content is such that the use of the richer mix leads to a very wet concrete.

(4) Strengths adequate for many practical purposes can be obtained with a vibrated 1-to-9 mix (by weight) with a comparatively low water-content, when a suitable grading is used.

(5) The effect of prolonged vibration of a wet mix is to throw out excessive water and to leave a concrete with a water/cement ratio less than the nominal value. This results in increased strength ; a higher strength would, however, be obtained by vibrating for a short time a mix whose water/cement ratio is equal to the value obtained with the wetter mix after prolonged vibration.

(6) There appears to be no important effect on the strength of concrete if it is subjected to short-period vibration subsequent to the initial vibration used for compacting it.
(7) Both the shrinkage and creep of concrete are reduced by using lower water-contents and compacting by vibration; the modulus of elasticity and the modulus of rupture are at the same time increased.
(8) The bond between concrete and steel is improved by using a lower water-content and compacting by vibration.
(9) The extensibility of vibrated concrete is apparently not less than that of hand-compacted concrete.

It is thought that the work so far carried out has shown that the use of vibration as a means of compacting concrete allows a lower water/cement ratio than would be possible for hand-tamped concrete; and that the physical properties of vibrated concrete differ from those of hand-tamped concrete only in so far as they are affected by the change in water/cement ratio.

REPORT OF THE WELDING PANEL OF THE STEEL STRUCTURES RESEARCH COMMITTEE.

This Report, recently published,[1] forms the last section of the work of the Steel Structures Research Committee. The Panel was formed in 1930 to undertake an investigation into the problems of the electric welding of steel structures. Reports on the progress of the research have appeared in the First and Second Reports of the Steel Structures Research Committee, but the research was not completed in time for the Final Report of the Committee, published in 1936.

The investigation may be classified as :—

(*a*) Statistical examination of the strength of welded joints.
(*b*) Investigation of non-destructive methods of testing welds.
(*c*) Research on the fatigue resistance of welded joints.
(*d*) Survey of existing published information on the design of welded joints.

From the statistical examination it is concluded that, in general, the quality of the electrode is more important than the workmanship of the welder, but that, confining attention to Grade A electrodes (that is, electrodes giving a minimum weld-metal strength of 28 tons

[1] Published by H.M. Stationery Office. Price 6*s*.

per square inch), workmanship of the welder becomes the more important factor. Much more consistent results are obtainable with butt-welds than with fillet-welds, but the occurrence of cracked fillet-welds is nevertheless very low. The ductility of butt-welds is much below that of ordinary structural steel. Whilst there is little difference between the average strength of horizontal and overhead fillet-welds, the strength of vertical fillet-welds is in general less.

In view of the desirability of developing a practicable commercial field-test for non-destructive routine examination of welds, X-ray, magnetic, electrical, and stethoscopic methods were investigated. By no method was it possible to obtain consistently reliable results, and the attempt to develop a satisfactory field-test was considered unsuccessful. Radiographic examination is of use in the case of butt-welds, but is unsuitable on the grounds of expense for field-tests except in the case of particularly important welds.

The fatigue-strength of actual weld-metal is definitely lower than mild steel of the same tensile strength. The normal static test of a fillet- or butt-weld is no indication of its strength under repeated loadings. Stress-concentration at the roots and toes of welds becomes of great importance. The tests emphasized the superiority of the high-class covered electrode over the bare-wire electrode. Nevertheless, tests abroad indicate that the fatigue-strength of a good welded joint is not necessarily less than that of the corresponding riveted joint. The possibility of fatigue-failure in welded connexions of steel floor-beams was found to be far too small to merit consideration, except in buildings in which heavy machinery rests upon the frames or in which marked impact-loading occurs.

A survey is given of existing published information on the design of welded joints under the headings :

- Part 1.—The strength of, and the stress-distribution in, welded joints.
- Part 2.—The influence of fatigue on the design of welded connexions.
- Part 3.—Residual welding stresses.
- Part 4.—Strengthening riveted joints by welds.
- Part 5.—Welded structural connexions.

Attention is drawn to the fact that little is known concerning the effect of residual stresses on the load-bearing capacity of a welded structure, and it is suggested that welding processes should be arranged so as to reduce such stresses to a minimum. In connexion with the strengthening of riveted joints by welding, the need is pointed out of subjecting the joints to sufficient load during the process to eliminate further slip under subsequent loading.

It is concluded that the investigations have produced much new information, which should form a useful basis for a code of practice for the use of electric welding in building construction. Such a code is being prepared by the Institute of Welding, at whose disposal the results have been placed.

RESEARCH WORK IN ENGINEERING AT LOUGHBOROUGH COLLEGE, FEBRUARY, 1938.

Civil and Mechanical Engineering.

A comprehensive series of researches is being made into the performance of the pelton wheel. The effect of modifications in bucket design, inclination of buckets, position of jet, and operating speed, have been studied stroboscopically and quantitatively. Several modifications have been made leading to increased efficiency of operation.

At the same time centrifugal-pump performance has been investigated, a careful study being made of the speed and direction of flow in the water passages by pitot-tube exploration. Mixed-flow and axial-flow impellers are being studied. A similar exploration has been made of the flow in pipe-bends.

Electrical Engineering.

A research—temporarily suspended—has been instituted into the effect of variations of temperature and humidity on the breakdown voltage for surface-discharge across high-tension insulators. High-voltage apparatus is available capable of producing voltages of 500,000 at ordinary power-frequencies, and 800,000 as an impulse voltage.

Chemical Engineering.

Researches of engineering interest are a study of the effect of adsorbed gases on the properties of metals, and a study of the effect of small additions of hydrogen, methane, acetylene, and ethylene to carbon monoxide on the rate of ignition of an explosive mixture. It is found that the presence of slight traces of these gases has a profound accelerating effect upon the rate of ignition. This research is of value in connexion with the gas-engine.

In addition, work is being carried out on the practical study of lubrication problems, and laboratory and engine tests are being correlated with the action of metal detritus in the ageing of compounded oils. Other problems being investigated cover the high-

temperature and pressure reactions of hydrocarbons, and other reactions with ammonia and similar substances.

Aeronautical Engineering.

A closed wind-tunnel 2 feet square, with possible wind-speeds of up to 100 miles per hour and an open-jet return-circuit wind-tunnel 12 inches in diameter have recently been installed, but no research has as yet been initiated.

The above researches are being carried out under the direction of Messrs. T. E. N. Fargher, Ph.D., M. Eng., Head of the Department of Civil and Mechanical Engineering; J. F. Driver, Head of the Department of Electrical Engineering; and G. M. Dyson, B.A., B.Sc., Ph.D., Head of the Department of Chemical Engineering.

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* When it is known that a reference will appear in an early issue of "Engineering Abstracts" this fact is indicated by an asterisk.

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OBITUARY.

HUGH HENRY GORDON MITCHELL, O.B.E., V.D., the eldest son of Captain H. H. Mitchell, of the Rifle Brigade, was born on the 7th November, 1874, and died in London on the 11th March, 1938. After completing his education at Bloxham School, he served a pupilage of 2 years under Mr. R. J. G. Read. In 1893 he went as an apprentice for 2½ years to Messrs. Heenan & Froude of Manchester, and worked for part of the time on the construction of the Blackpool and Wembley towers. He was subsequently in executive charge of the erection of 5,000 tons of bridge work on the Nottingham section of the Great Central Railway. Later he was employed by Messrs. Coode, Son & Matthews on the Prince of Wales pier, Dover, 1898–1901; on the Outer Barrier works, Hodbarrow, 1901–1903; and at Colombo harbour, 1903–1905.

In 1905, Mr. Mitchell was appointed Harbour Engineer at Madras, and in 1919 succeeded the late Sir Francis Spring as Engineer Chairman of the Madras Port Trust. He was Vice-Commodore of the Sailing Club, which he was instrumental in forming, and commanded the Madras Artillery Volunteers. He came to England in 1916 to join the Army, but great damage to the harbour at Madras due to a cyclone necessitated his recall. He returned to England in 1917 and joined the Royal Artillery with the rank of Colonel. In 1919, he was appointed Honorary A.D.C. to the Viceroy.

From 1921 until his death Mr. Mitchell was a partner in the firm of Coode, Fitzmaurice, Wilson & Mitchell (later Coode, Wilson, Mitchell, & Vaughan-Lee), Chartered Civil Engineers, and was principally engaged on the design and supervision of important dock and harbour works in Great Britain, Kenya, Uganda, Zanzibar, Tanganyika, Ceylon and the Straits Settlements, and irrigation works in Iraq, to which countries he paid frequent visits. He was awarded the Brilliant Star of Zanzibar in 1929.

Mr. Mitchell was elected a Member in 1910, and from 1920 to 1922 served as a Member of Council for India. He was the recipient of an Indian Premium in 1912 and of a George Stephenson Gold Medal in 1918 for Papers dealing with harbour works at Madras.

He was a Lieutenant-Colonel in the Engineer and Railway Staff Corps, in which he had served 13 years. He married, in 1926, Dorothy May Mitchell, who survives him.

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